WATER POWER

A technical journal devoted to the study of all aspects of Hydro-electric developments

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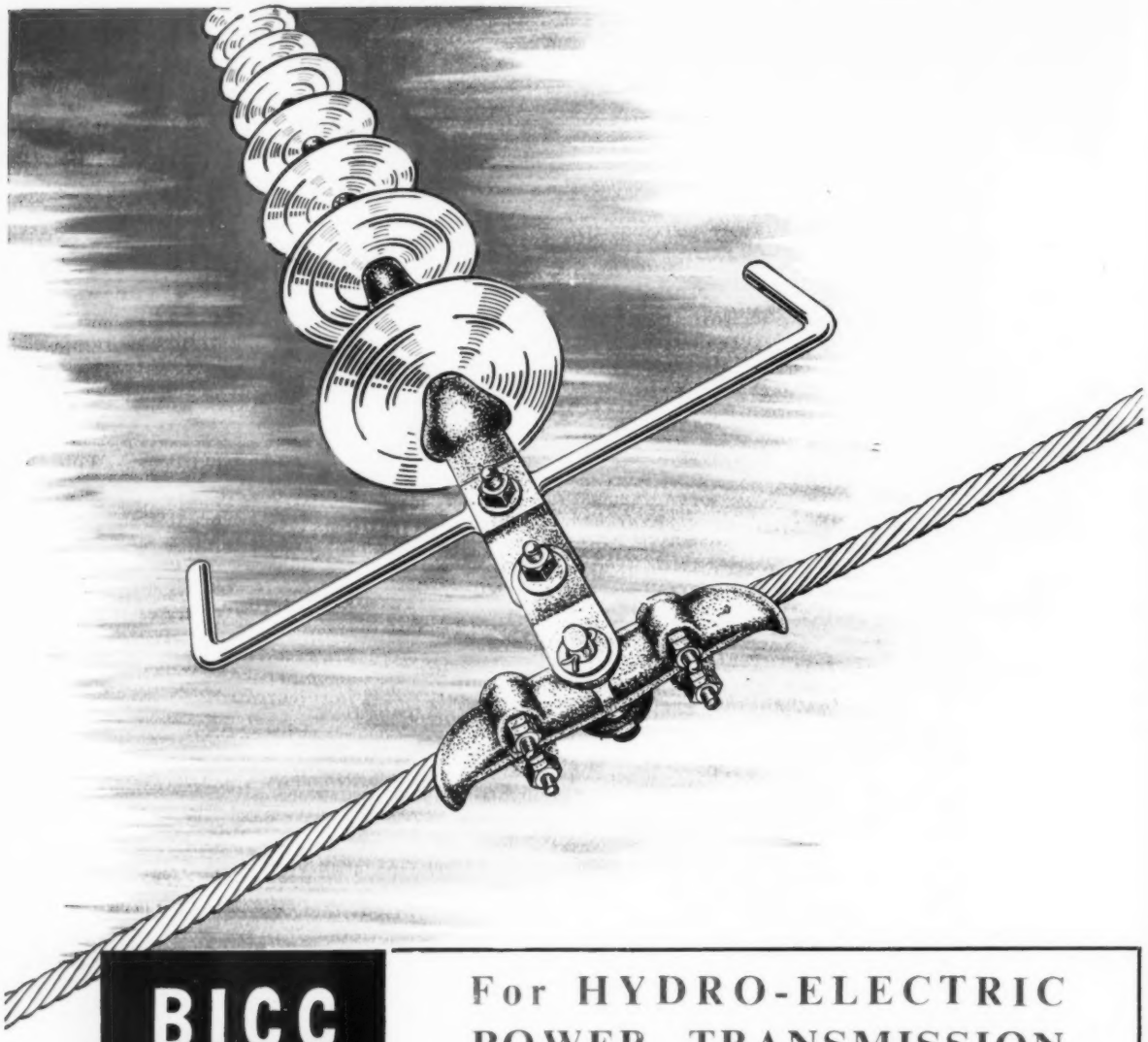

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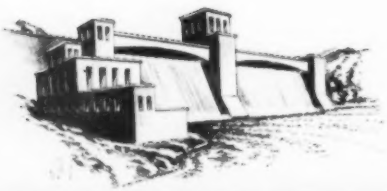

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WATER POWER

A technical journal devoted to the study of all aspects of Hydro-Electric Development

The Hydro-Electric Outlook

IT is a long time since we have read so objective a survey of the hydro-electric scene as that which appears in the paper presented to the 1954 British Electrical Power Convention by Sir George H. Nelson, the chairman and managing director of the English Electric Co. Ltd. He takes a conservative estimate of the world's hydro-electric resources, at a figure of between 600,000 and 700,000 MW, and adds that the proportion developed at the end of 1952 is estimated to be 90,000 MW.

His next figure in the logical chain of argument he builds up in surveying the future of hydro-electric power, is that which relates to the rate of increase of electricity consumption in the world as a whole; and here our readers may, perhaps, take leave to form their own ideas, rather than to accept the average figure of 7 per cent. compound increment put forward in Sir George's paper. The rate of increase in electricity consumption in different parts of the western world varies widely from about 4 to 14 per cent. per annum—but there is a factor which leads us to suggest a somewhat higher figure than 7 per cent. as being more realistic. Nowadays any new consumer, whether he is connected to the urban mains in a highly industrialised country or to a recently erected transmission line in the wilds of Africa, steps in to the electrical scene at a different level than that of his predecessors. Twenty years ago a new consumer had a limited potential demand, since he only used electrical energy for a limited range of purposes, few of which went beyond his main power requirements. Now the pressure of industrial competition has made him (if he is an industrial consumer) fully conscious of the need to start from the very beginning at a high level of mechanisation—and mechanisation has come to be synonymous with electrification. Again, if he is a domestic consumer, the enormous range of electrical appliances available to him will mean that he may very likely plan an all-electric house, although previously his usage of electric current was probably confined to lighting only. But even Sir George's 7 per cent. compound means the doubling of the installed power approximately every ten years. "Thus, from the installed capacity of 90,000 MW estimated for the end of 1952, we should reach 180,000 MW by 1962; 360,000 MW by 1972; and 720,000 MW by 1982, whereas the hydro-electric power at present known to be capable of development is only 591,000 MW in those parts of the world which we have been considering." *

Emanating from such a source these arguments are weighty, but it appears to us that two factors affecting this conclusion have been overlooked. First, the development of any form of electricity generation—thermal power stations fired by coal or oil or even by heat from nuclear piles—can never provide of themselves an effective solution to the greatest problem that the supply engineer has always had to face, viz., the problem of meeting peak loads, or conversely, of storing off-peak energy. In this connection, it is only the hydro-electric engineer who has any kind of solution to offer. Pumped-storage schemes and possible tidal schemes are the two factors to which we refer.

It is true that pumped-storage schemes can only be operated in suitable localities; but this limitation also applies in increasing measure to ordinary hydro-electric schemes. The improvements which have been slowly but surely apparent in regard to the harnessing of very high and very low-head projects, the possibilities of variable-speed alternators, and the efficiency available from modern methods of power transmission, have combined to make many hydro-electric sites which were previously thought to be uneconomic or impracticable, to become commercial propositions.

In the same way, it may well happen that re-examination in face of changed circumstances may bring a different verdict on the practicability of many pumped-storage schemes. Indeed this process is already in being in Great Britain, where a 300

* Excluding China and the U.S.S.R.

ESTIMATES OF HYDRO-ELECTRIC POWER

Country	Energy in kWh p.a.		Power in MW		
	Ultimate developable (millions)	Developed to end of 1952 (millions)	Ultimate developable (millions)	Developed to end of 1952	Percentage developed
North America	807,500	175,000	161,500	32,500	20
Europe (excl. Russia)	491,100	133,646	98,050 20,000	29,625 5,000	30 25
Russia in Europe	90,000	20,000	25,000	3,000	12
Siberia and other U.S.S.R.	111,000	13,200	88,000	10,280	12
Asia (excl. U.S.S.R and China)	454,000	44,300	50,000	500	11
China	250,000	2,500	5,560	605	11·1
Australia	24,500	3,000	15,000	10	0·1
Papua and New Guinea	66,000	450	8,000	800	10·1
New Zealand	40,000	3,400	100,000	300	0·3
Africa	450,000	1,320			
East Indies, Pacific Islands, etc.	66,000	4,500	15,000	1,000	6·7
South America	450,000	27,000	100,000	6,000	6
TOTALS	3,300,100	428,316	686,110	89,620	13

MW pumped-storage station, once thought impracticable or uneconomic, is being planned for North Wales. It may also be that the coming of atomic power generation will emphasise the need for storage. The British Minister of Fuel and Power, in the House of Commons recently, stated that Britain's future atomic power stations would be operated "flat out at full load day and night, as far as possible, in order to save as much coal as possible." It may well be the case that the shutting down of an atomic reactor every night will pose serious problems for the operating engineers; and in any case, the cost of the shielding devices is so great that only very large units can be contemplated as a permanent feature in the future power landscape. The very large base-load atomic stations would thus have a still greater need for ancillary generating equipment to carry the inevitable peaks. In consequence it appears likely that the hydro-electric engineer of twenty-five years hence will be employing his skill in devising very large pumped-storage schemes, in enlarging high-head reservoirs, or in utilising the topography of the terrain by methods not at present regarded as being practicable.

Not least among the important data presented by Sir George (much of which related to the other branches of electrical engineering) is the appended table, which gives an estimate of hydro-electric power, potential and developed, in all parts of the world. "Estimates of this form of power," Sir George added, "must of necessity be largely guesswork, especially for certain parts of the world, but the figures given are tempered by first-hand knowledge of certain areas and judgment derived from experience."

St. Lawrence River Model

IN order to study the effect of channel improvements on water levels and the distribution of currents, the National Research Council is constructing a model of the St. Lawrence River in the vicinity of Cornwall. The scales chosen are 1 : 96 vertically and 1 : 480 horizontally, which will produce a model about 100 ft. long and 25 ft. wide. It is being built up of pre-cast concrete blocks, 2 ft. square, which are being cast from patterns made directly from the contour map. This arrangement, it is pointed out in the Quarterly Bulletin (No. ME 1954(1)) of the National Research Council of Canada, has the double advantage that the model may be built before the space to house it is available and will also greatly facilitate detail changes.

Brazilian Traction Report

IN its Annual Report for 1953, Brazilian Traction, Light and Power Co. Ltd. states that river flows and run-offs during 1953 in the watershed areas of the Rio system were the lowest recorded in over 30 years. In spite of the critical water situation, the Rio system generated 2,269,559,100 kWh in 1953, an increase of 12·1 per cent. over 1952. The increased generation resulted partly from the bringing into service of two 35,000 kW units of the Nilo Peçanha (formerly called Forçacava) underground generating plant, thus increasing rated generating capacity by 70,000 kW. With this additional capacity, it was also possible to operate the Paraíba-Piraí Diversion pumping units continuously for the balance of the year, thereby partially relieving the critical water shortage. Favourable progress was made during the year towards the installation of the four 65,000 kW units of the Nilo Peçanha plant, all of which were scheduled for operation in 1954. The third set of pumps was placed in operation at Vigário on August 27 and at Santa Cecilia on September 30. The installation of the fourth set of pumps was begun during 1953 and they were scheduled to be available for operation in May, 1954.

The serious shortage of water in the São Paulo area became worse in 1953 and was even more critical than in the Rio area. Rainfall in the run-off areas of the reservoirs of the São Paulo system was considerably less than had been anticipated and river flows were 30 per cent. below average. Because of the lack of generating capacity and the critical water storage position of the São Paulo system, the public authorities sanctioned the disconnection of distribution feeders as a means of reducing the load on the system. Total generation on the São Paulo system was 3,503,533,700 kWh, a decrease of 2·7 per cent. from 1952. At the Itupararanga plant and the small run-of-river plants output was reduced owing to a lack of water. No generating capacity was added to the São Paulo system during the year.

During 1953 construction work on the Piratininga steam-electric plant and the Cubatão underground hydro-electric plant progressed satisfactorily. The two generating units of the Piratininga plant with a total rated capacity of 160,000 kW (but with a capability of 200,000 kW) are scheduled to go into service during the latter part of 1954. It is hoped that two of the four generating units of the Cubatão underground plant can be brought into operation during 1955.

Loch Shin Scheme

THE North of Scotland Hydro-Electric Board's Shin scheme will involve the construction of five power stations which will use the waters of Loch Shin and tributaries, the River Cassley, the River Brora and other streams in the County of Sutherland. The catchment has an area of about 250 sq. miles, and the total cost is estimated to be about £8,800,000. Four dams will be necessary and the five power stations will have a total capacity of 40 MW.

Loch Shin will be enlarged by a dam near Lairg, in which will be incorporated a generating station having an installed capacity of 3,500 kW. A second

dam will be built across the River Shin about a mile below the main dam and from the reservoir thus formed, water will be led by an aqueduct to the main generating station near Inveran which will have an installed capacity of 23,000 kW. Both dams will include provision for the passage of fish. Water from the upper River Brora will be diverted by a weir at Dalnessie and led by an aqueduct into a tributary of the River Tirry. The Cassley station, on the southern shore of Loch Shin near the western end, will use water brought by tunnel from the upper Cassley and its contributory streams.

There are two subsidiary developments to the north-west of Loch Shin. One consists of a dam to enlarge Feur Loch, and an aqueduct connected with a power station having a capacity of 1,000 kW. This station is situated on the southern shore of Loch Merkland near Garvault, and a second station, also on Loch Merkland, but on the opposite side, will have a capacity of 5,000 kW. George Wimpey & Co. Ltd. have been awarded the contracts to build the Shin generating station and tunnel, and Lairg dam. The tunnel will be five miles in length and the dam 38 ft. high and 1,370 ft. long. The contract to construct the Cassley generating station on the south shore of Loch Shin near Creanich, the aqueducts to collect water from the headwaters of the River Cassley, and the 2½ miles long tunnel which will lead it to the power station, has been entrusted to R. J. McLeod (Contractors) Limited of Glasgow. The total value of these contracts is in the neighbourhood of £5,000,000.

Snowy Mountains Generators

THE two generators recently ordered from ASEA (Australia) Limited by the Snowy Mountains Hydro-electric Authority for the Upper Tumut scheme are believed to be the largest ever ordered for erection in the Southern Hemisphere. They are rated at 84,200 kVA, 0·95 power factor, 12·5 kV, and will be coupled to English Electric turbines running at 375 r.p.m. Although the order was placed with the Australian company the machines will be built in the ASEA works at Västerås, and are due for delivery by January 1957.

These generators will be the sixth and seventh to be built by ASEA for Australasia. The other machines, also to be coupled to English Electric turbines, comprise two 25,000 kVA, 0·8 power factor, 11kV, 375 r.p.m. generators recently completed for Trevallyn, Tasmania, and three 23,333 kVA, 0·9 power factor, 11kV, 125 r.p.m. generators on order for Atiamuri, New Zealand.

Stornorrfors Progress

PRELIMINARY work is now well in hand on the Swedish State Power Board's Stornorrfors scheme, although operations on the Ume river, which were commenced in September 1953, have so far been devoted mainly to the erection of offices, workshops, construction camp, and preparation of the site.

Stornorrfors will be the largest hydro-electric station in Sweden, having a capacity of 375 MW as compared with the 350 MW at Harspränget, which is the largest at present in operation. It is believed that the tailrace tunnel, which will be 4km. long, will have the largest cross section of any single tunnel in the world. Its 360 sq. m. section represents a considerable increase on that of the supply tunnel at Kilforsen, which is 208 sq. m. and is at present the largest in Sweden. In deciding upon the technique to be employed in driving the tunnel, the Power Board is collaborating with Atlas Diesel A.B., Stockholm, who will be supplying the equipment.

Three 178,000 h.p. 125 r.p.m. Francis turbines have already been ordered from Nydqvist & Holm, A.B., Tröllhattan. These machines, which will be amongst the largest in the world, will have a runner dameter of 5·15 m. and will operate under a head of 73 m. and a maximum discharge of 200 cu. m. per sec. The alternators for these sets have just been ordered from ASEA, Västerås, and will be rated at 150 MVA at 0·9 power factor, 125 revs. per min.

Congress on Large Dams

THE Fifth International Congress on Large Dams, organised by the International Commission on Large Dams of the World Power Conference is to be held in France and North Africa from May 31 to June 17, 1955, and Bulletin No. 1 covering this Congress has now been issued by the French National Committee. The Congress proper will be held in Paris from Tuesday, May 31 to Saturday, June 4, after which there will be two alternative study tours in France from June 4 to 9. From June 11 to 17 a choice is available from four study tours in Algeria, Tunisia and Morocco. Prior to the Congress, from May 25 to 27, a meeting will be held in Paris organised by the French Committee on Soil Mechanics.

Details of the questions to be discussed at the Congress were given in our Editorial Note on page 400 of our November 1953 issue.

Copies of the Bulletin, together with provisional registration form, can be obtained through the respective National Committees.

International Power Co-ordination

PARALLEL with the developments which are taking place in the co-ordination of European coal and steel production an international organisation for the development of hydro-electric power is under consideration. In December 1952 a body known as "The Study Company for the Development of Alpine Water Power in Austria, Limited" was formed, but its scope was limited to exploiting the resources of the Austrian Alps with only a small number of States participating. It is now hoped, however, that it may form the nucleus for a wider international co-operation towards the balanced development of coal and hydro-electric power production. The outstanding problem in respect of hydro power development is that of the high capital investment involved, and in this matter the projects envisaged for Austria alone far surpass the financial resources of the four countries participating in the Study Company, viz., France, Italy, Germany and Austria. Attempts are being made to enlist the active interest of the International Bank for Reconstruction and Development, which could arrange for the necessary finance. Problems involving proprietary rights, power generation and deliveries are being considered by working committees.

Fig. 1. View of mountains in Calgary area

Hydro-Electric Power in Alberta

An account is given of the activities of Calgary Power Ltd., with special reference to the new Spray River plant and the scheme of automatic and supervisory control which has been introduced.

CALGARY POWER LTD. is the largest producer of power in the Province of Alberta and operates a group of eight hydro-electric plants on the Bow River and its tributaries, together with a transmission network extending to over 3,500 miles. Until 1951 the total horsepower of the hydro plant installed was 117,000 and as the system load was increasing at the rate of 15,000 kW per annum there was a growing necessity to provide additional capacity. The potentialities of the Spray River had aroused interest as long ago as 1911 as the topography of the country is peculiarly suitable for the installation of a high-head plant. Its headwaters lie in the Continental Divide about 25 miles south of Banff, towards which it flows in a characteristically glaciated valley which, opposite Canmore, approaches within a few miles of the Bow River situated 1,205 ft. below. By constructing a storage dam on the Spray two advantages are secured; in the first place a direct increment of 88,600 h.p. is gained by a 1,285 ft. head which is utilised in three plants situated near Canmore, and in the second, the storage reservoir built in conjunction with this development is of great benefit to the performance of the plants operating on the lower stretches of the Bow River.

Before the present dam was built to impound the waters of the Spray two small natural lakes, 400 acres in extent, already existed, and these have been incorporated in the new storage dam which covers an area of 4,800 acres. Between the Spray River and the Bow River there is a further tributary, Goat Creek, which also flows northwards to join the Spray about 11 miles distant. Both valleys, which are separated by a relatively low divide, are at roughly the same level and by erecting a dam in the Spray the water was induced to flow down Goat Valley where a further

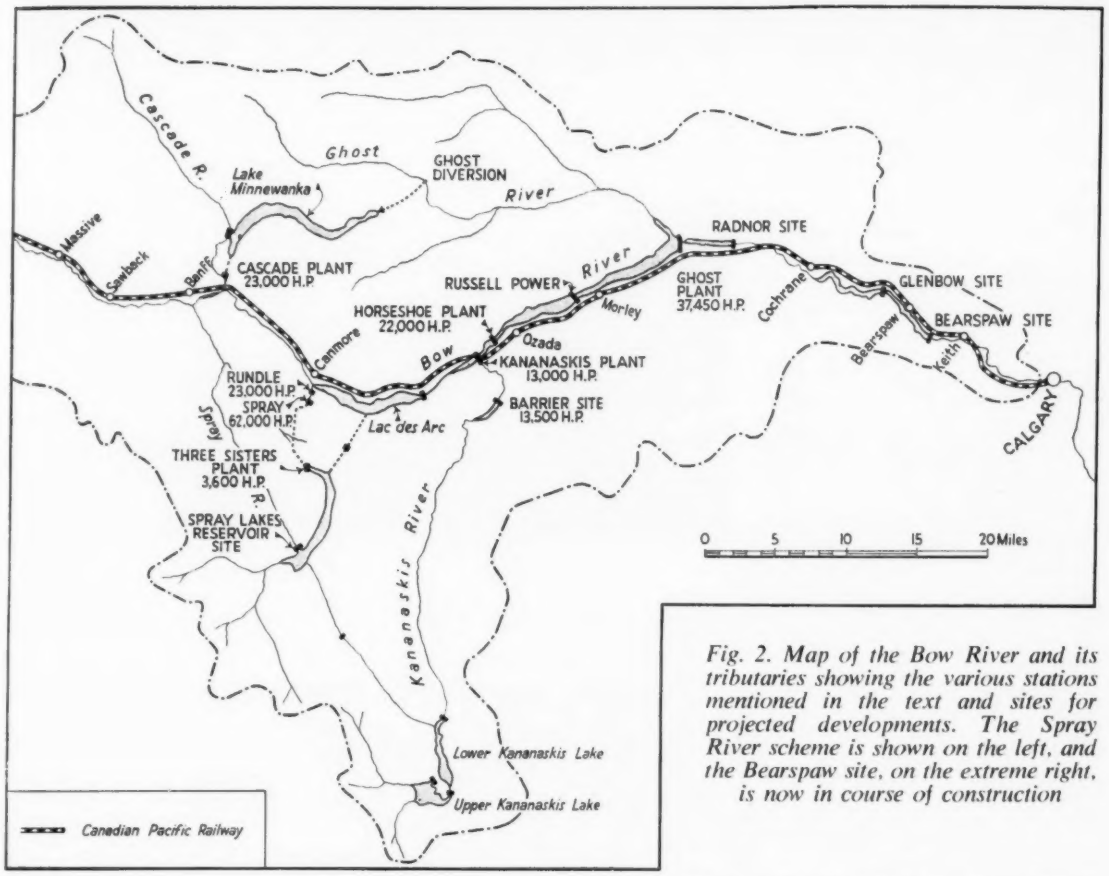


Fig. 2. Map of the Bow River and its tributaries showing the various stations mentioned in the text and sites for projected developments. The Spray River scheme is shown on the left, and the Bearspaw site, on the extreme right, is now in course of construction

dam was constructed to provide a live storage reserve of 180,000 acre feet. At the site of the diversion dam at the Spray River the elevation at water level was 5,420 ft. The elevation of the pass leading to Goat Valley was 120 ft. above this level and to unite the two valleys and at the same time produce an adequate storage, the Spray diversion dam was constructed with the full supply elevation at 5,583 ft., thus raising the level 163 ft. As this elevation is 40 ft. above the natural level of the Goat valley, a further dam was necessary. This structure is called the Three Sisters dam from its proximity to the well-known mountain of this name. The slope of Goat valley floor is relatively flat, so that the lower regulation level was largely determined by the elevation of the pass, as a greater drawdown would have necessitated an excessive amount of excavation in the connecting canals. In view of this consideration the lower regulation was set at 5,527 ft., necessitating the construction of 13,000 ft. of canal. The floor of the canal at Three Sisters

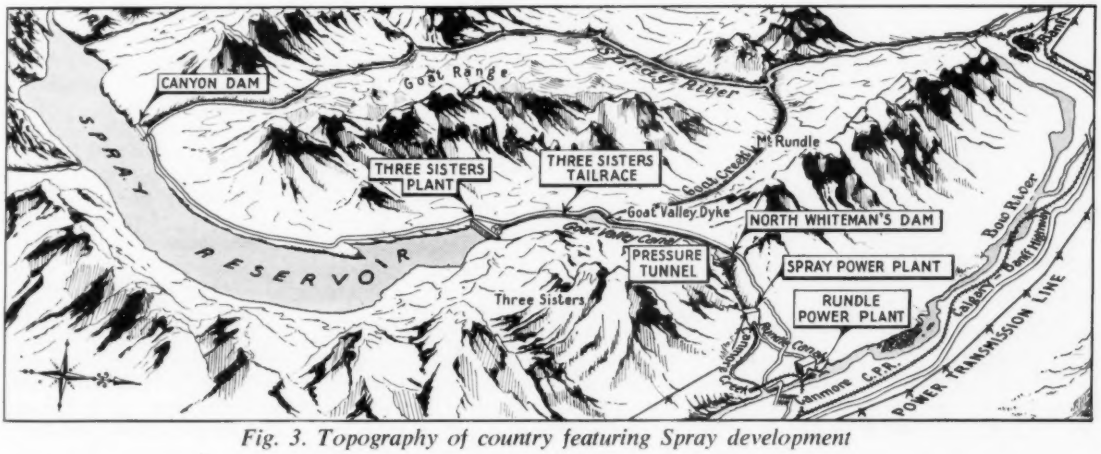


Fig. 3. Topography of country featuring Spray development

dam is 18 ft. below the original ground level, and as this head would otherwise have been lost a 3,600 h.p. plant was installed here. Although this plant is not economical when considered from the purely power standpoint, it forms a very desirable means of controlling the flow to the canal system. It is only available, of course, when the reservoir elevation is reasonably high, and the water is simply bypassed when the level is low.

The discharge canal from this unit is led along the eastern side of Goat valley to a 270 acre regulating pond and then follows a northerly direction towards Whiteman's pass. Here the water flows through a tunnel which penetrates a spur of the mountain, at the exit end of which there is a further pond, 27 acres in extent, created by a small earth embankment known as the North dam. From this pond a short canal cut in rock leads to the Spray plant intake works, the water then passing into the pressure tunnel which first descends for a distance of 1,250 ft. at an inclination of 45 deg. and subsequently follows a level course, 915 ft. long, to the power house. This structure is built against the rock face and is equipped with a 62,000 h.p. Francis turbine by the Dominion Engineering Company, vertically coupled to a Canadian Westinghouse 47,000 kVA 450 revs. per min. alternator, the head being 900 ft. As the total head between the Spray plant intake and the Bow River is 1,220 ft., the remaining drop of 320 ft. is utilised in a further power house, the Rundle, on the Bow River. The Spray turbine discharges into a pond formed by damming the creek which drains Whiteman's pass and from here the water is led by a dyke and canal to the Rundle intake, about 1½ miles away. From this intake the water is taken by a steel penstock, 1,300 ft. long, to the Rundle power house, which is equipped with a 23,000 h.p. Francis turbine.

Fig. 4. Spray lake at raised elevation, 5,583 ft.

Major Constructional Works

The canyon diversion dam erected across the Spray valley is an earthfill structure having a core of rolled glacial till and graded gravel embankments. The crest is 20 ft. wide and the upstream and downstream faces are respectively at slopes of 3½ to 1 and 2¼ to 1. To protect the upstream face against wave erosion it has been covered with a 2 ft. thickness of truck-placed rip-rap carried somewhat below the minimum regulation level. The maximum height of the dam from the rock foundations to the crest is 192 ft.

To test the foundation conditions some 2,880 ft. of cores were obtained by diamond drilling and although there is a thrust plane along the west side of the canyon at the dam site, which has resulted in a certain amount of brecciation, this broken ground has subsequently been naturally cemented and was not considered to be likely to give trouble. The permeability all along the site is low and the only disturbing circumstance that was encountered was that several of the diamond drill holes struck artesian water. One of these had a flow of 30 g.p.m. under a static head of 35 ft. and other springs were laid bare as rock was stripped off and cleared. It was clear, however, that this water had a deep-seated origin and had no connection with the flow of the river.

Before the core was laid more downstream springs were collected in finger drains built of graded sand, protected by a shell of cement mortar, and extended to a sump at the downstream edge. From this sump the water is drained away by two 6 in. pipes to the toe of the dam. In the upstream portion of the core the springs were collected in 16 in. dia. risers from which the water was pumped continuously. These risers were increased in length as the height of the dam increased until a point was reached when the water no longer reached the top. The risers, which were seven in number, were then tamped with puddled clay and have given no further trouble. Observations of the 6 in. drains from the collecting sump have shown a constant flow of 150 g.p.m., there being practically no variation in the quantity whether the reservoir is full or drawn down. The diversion tunnel was constructed in the east bank of the canyon and was 1,030 ft. long and had a cross section 16 ft. square. While the tunnel was being driven, the river bed was cleared of overburden, a good deal of the gravel being dredged by draglines. These operations took place in the winter months and as the flow is very low in this period, dropping down to 100 and even 60 cusecs, the construction of the cofferdam was a simple matter. Two 48 in. dia. corrugated iron pipes were placed in a trench about 20 ft. west of the river, which was then dammed with fill until the water flowed through them. The fill was then built up to such an elevation to enable the spring floods to be carried by the diversion tunnel without overtopping. The fill was rendered impervious by a till facing on the upstream slope, the whole being so located that it could be incorporated in the final dam. Once the diversion tunnel was completed the 48 in. pipes were plugged and the water diverted into the tunnel. As the rock in the tunnel contained several beds that were not absolutely water-

tight it was decided to fill in the entrance of the tunnel in addition to inserting a concrete plug. The concrete plug was traversed by two pipes equipped with valves and these provided a means of ascertaining whether the fill covered the portal effectively. At first there was some tendency for pressure to build up behind the clay fill at the entrance but it has now become consolidated. The dam itself was completed in a period of five months, the million cubic yards of material required for its construction being obtained from nearby borrow pits.

The Three Sisters dam at the north end of the reservoir is also an earthfill structure and is 2,200 ft. in length. At this site the valley floor is covered with glacial till overlain by gravels and sand to a thickness of 110 ft., some of the beds being compressible. One such bed, a lens of silt 20 ft. thick and lying 75 ft. below the surface, was estimated to occasion 0·9 ft. of settlement at the centre line of the dam in the section containing the outlet works, and other complications arose through some of the gravels above this bed being extremely porous.To meet the anticipated settlement—which, in fact, was not realised—it was decided that the 10 ft. dia. mild-steel conduit through the dam should be provided with flexible expansion joints. This pipe was laid in a trench excavated for the purpose and backfilled with well-compacted impervious material. Every precaution was taken with the concrete intake structure which was also carefully compacted with impervious material which was carried some distance beyond the front apron and extended on either side to blend with the impervious blanket covering the upstream face of the dam. Near the middle of the 10 ft. conduit there is a cylindrical steel and concrete tower which houses the head gate and its operating mechanism.

At the power house the pipe bifurcates into a symmetrical Y-piece, one leg feeding the turbine and the other being equipped with a diverging tube to bypass water direct into the canal. The power house was built on a raft foundation, strongly reinforced to ensure that if settlement did occur the whole structure would settle as a unit. A good deal of seepage was noticed in the slopes of the tailrace cut on either side of the power house and as a safeguard against the movement of fines a thick inverted filter was placed in the tailrace downstream from the draught tube. This filter was graded from sand at the bottom to heavy riprap at the top and in addition the power house was surrounded by a closed curtain of steel piling to a depth of about 20 ft. The space between the piling and the building was filled with concrete, care being taken to prevent the concrete from bonding with the steel sheeting. Grout pipes, 10 ft. apart and 2 in. dia., were left projecting through the concrete slab and extended to well below the foundation. The object of the piling was to divert any flow of water from beneath the power house where it might conceivably erode the

Fig. 5. Spray lake showing Canyon dam

finer material and the purpose of the grout holes was to provide a means of righting the building, should any tilting occur, by injecting grout under the settled portion. As yet, however, the power house and surrounding area have proved to be quite stable and the grout pipes have only been used for observing the height of the water table beneath the power house.

At first it was intended that the dam should be built of run-of-pit material, with an impervious blanket covering the upstream face and extended well up the valley floor. In the course of excavating the canal, however, a deposit of high impermeability was found and this was used in preference to the material obtainable from pits. In consequence the dam was built as a virtually homogeneous structure, the coarser material being used downstream and the finer reserved for the upstream section. Only the portion adjacent to the intake structure and pipe was built in accordance with the original design.

It should perhaps be stressed that the Three Sisters dam is built across a valley 2,000 ft. wide which is covered with extensive beds of clean gravel and sand. Some seepage is thus inevitable; moreover much of the canal is built on top of a mass of talus and gravel under which run-off water from the mountainside can easily percolate. To conserve this water two pumping stations have been installed in Goat creek to return it to the canal. The upper station is equipped with a 700 h.p. pumping unit which is capable of handling 70 cusecs against a head of 80 ft., this flow being discharged to the canal by a 42 in. dia. wood-stave pipe 1,800 ft. in length. Operating conditions in the lower station require the handling of 40 cusecs against a 227 ft. head and a 1,400 b.p. motor is necessary. The overall gain secured by these measures will be evident when it is considered that at the cost of being lifted a maximum of 227 ft. this conserved water is first utilised in the Spray plant at an effective head of 875 ft. and then in the Rundle plant with an effective head of 300 ft. The net recovery after deducting the power consumed by the pump motors and taking the efficiency of the turbines and generators into consideration is estimated to be 75 million kWh per annum. Both pumping stations are automatic in operation and remote controlled and so require a negligible amount of attention.

The canal connecting Three Sisters and the Goat Pond is 1½ miles in length. In this stretch the valley is from two to three thousand feet in width and was at one time occupied by a glacier which flowed northward towards Banff but threw off a spur to the east which made its way through Whiteman's pass down into the Bow Valley below. The valley floor is remarkably flat for a distance of 8,000 ft. below the Three Sisters dam, dropping only 20 ft. in this distance. Beyond this point the gradient steepens until it is 200 ft. below the ground level at Whiteman's pass. In consequence, in its course along the valley the canal has to follow a route which becomes progressively higher above the level of the creek and as the valley is U-shaped the canal route is carried in ground that becomes progressively steeper. It was this circumstance which determined the upper limit of the canal elevation. Goat dyke is located at the point where the valley steepens and unfortunately, as the floor here consists of porous material, there was little chance of building a watertight structure. The pond flows into an earth canal 15,000 ft. in length, designed to carry 700 cusecs at a maximum velocity of 1·6 ft. per sec. The route of this canal follows the side of a hill having a transverse slope ranging from zero to 22 deg. to the horizontal, the ground consisting of alluvial detritus, glacial till, coarse limestone talus and ridges of solid rock. To make the canal impervious

Fig. 6. Goat valley dyke and Three Sisters plant

Fig 7. Spray plant, showing headworks at top of Whiteman's pass

a compacted clay lining was applied, this lining being spread to a depth of 2 ft. down the slopes and along the bottom with a bulldozer. It was then compacted with a sheepsfoot roller pulled up the slopes by a dragline. All the compaction work was closely supervised. Unfortunately a failure occurred over a length of 800 ft. where the canal was carried through a length of talus, which, it was subsequently learned, was underlain by a considerable mass of fine, uniform silt. Evidently underground water had seeped into this silt, possibly from leakage at the junction between the rock face and the floor lining of the canal, converting the silt into quicksand which yielded and flowed into the valley, carrying the embankment along with it. An exploratory bore at the mouth of the pass adjacent to the slide had not revealed any silt, and it had not been thought necessary to put down test holes into the talus which might be expected to drain well. In view of this collapse it was decided to abandon this section of the canal and replace it by a tunnel driven through the rock spur forming the south wall of the pass.

Some leakages occurred when the canal was filled but none was considered serious enough to necessitate draining out the water. Some were dealt with by inserting impervious plugs but in general this seepage problem was solved in a very simple and ingenious way. A drag, consisting of a 12 in. H-section girder, was made partially buoyant by lashing two logs to it and drawn along the sides and bottom of the canal. This stirred up the lining material and by adding a further amount of fine sandy silt as the beam was drawn along the leakage was reduced to a negligible amount at a very low cost. The drag is so designed that when submerged it exerts a pressure of slightly less than 1 lb. per sq. in. on the lining.

The Pressure Tunnel

The pressure tunnel was driven through mid-Devonian strata consisting of thick-bedded and thin-bedded limestones, dolomite, some argillite and a few thin bands of shale. The site chosen for the Spray plant was 1,800 ft. horizontally from the intake and 915 ft. of this distance is occupied by a tunnel graded at one per cent. rise from the power house. It then rises at an angle of 45 deg., as has already been mentioned, to emerge at the short canal leading from Whiteman's pond. The final 350 ft. of the horizontal section is furnished with a steel lining, this length carrying the tunnel to a point at which there is a minimum of 216 ft. of rock cover. The thickness of the steel lining was graded inversely in proportion to the depth of cover, being $\frac{7}{16}$ in. thick at the inner end and $1\frac{3}{8}$ in. at the portal. The diameter was also progressively reduced from 7 ft. 6 in. at the upper end to 4 ft. 8 in. at the turbine valve. Above this section the tunnel is lined with concrete to a finished diameter of 8 ft.

The tunnel was driven from the downstream portal only and was roughly of horseshoe cross section, 11 ft. high and 10 ft. wide. About 500 ft. up the rising portion several feeders of water were encountered and had to be sealed by grouting. This was done by driving a small heading into the side of the face where the leakage was least and drilling grouting holes laterally from it to intercept the fractures. After this experience diamond drill holes were maintained 50 to 60 ft. in advance of the face and grouted wherever any water was encountered. As an additional precaution, before slashing the pilot heading to the full dimensions, a grout curtain was injected all around and beyond the fractured zone. The wet section extended about 150 ft. and took 40 days to penetrate, but beyond this point no further trouble was encountered.

After the tunnel was holed through, more grouting was done to reduce the seepage to a point at which the concrete lining could be safely poured. Good results were obtained excepting over a length of 140 ft. where the fractures were so fine that they refused to seal. A total flow of 200 g.p.m. issued from these cracks and special measures had to be adopted for

the lining. A steel pipe, made of $\frac{3}{8}$ in. plate and stiffened on the outside by ring girders, was inserted in the wet area in lengths of 20 ft. Each length was equipped with jack screws around the circumference and the pipe lined up and securely held by these means. The sections were also fitted with coupling bands to ensure reasonably tight joints. The grout was injected through holes in the steel casing, equipped with threaded plugs, the whole of the space between the lining and rock walls being filled with concrete. The steel lining was not designed to resist hydrostatic pressure acting on the outside—a condition which might easily arise if the penstock were to be emptied sufficiently quickly to leave a build-up of pressure in the surrounding strata. To meet such a contingency a perforated galvanised pipe, 6 in. dia., was laid on each side of the steel lining, next to the rock. These pipes were wrapped in heavy brown paper to prevent the entry of mortar during concreting and were carried down to the tunnel outlet to discharge into the tailrace.

Where the walls were dry the concrete was fed to the forms by gravity through a 6 in. dia. pipe which extended to the full depth of the inclined portion. In the level portion the concrete was at first handled by a pneumatic placer, but this was subsequently replaced by a Pumpcrete unit assisted by a pneumatic booster near the point of delivery.

Fig. 8. Spray power house with Howell-Bunger valve in operation

The Rundle Plant

The Rundle pond provides a sufficient volume to absorb all normal fluctuations resulting from the operation of the two plants, but in the event of a sizable shutdown at Rundle a syphon spillway, consisting of two tubes, affords a means of bypassing the flow from the pond to Canmore creek. Each tube has a capacity of 350 cusecs and the twin discharge takes place over a steep rock at a point about $1\frac{1}{2}$ miles from the confluence of the creek and the Bow River. This length of the creek bed is steeply graded, having a total drop of 260 ft. to the Bow. Canmore creek drains an extremely limited area and even its flood flow has a very modest volume, altogether insufficient to disturb the stability of the large amounts of gravel and wash material which occupied the valley floor. It was anticipated that the sudden discharge of 700 cusecs from the syphons would cause heavy erosion but no alternative was possible. As it happened the Spray plant was put into commission two months before the Rundle station was completed and the syphons were put into full operation during this period. The resulting erosion was of a spectacular order and according to Mr. Eckenfelder*, provided a fascinating demonstration of the sort of action that takes place in a river valley in the course of centuries. Immense quantities of material were swept down the valley and created a pressing problem of removal at the point where the creek enters the Bow. Three draglines and six bulldozers had to be thrown into commission to remove the sand and gravel which threatened to engulf houses, the railways and roads, and altogether some 200,000 cu. yds. of material was heaped at the entrance to the Bow before the Rundle plant was put into commission.

(To be continued)

* "Spray River Hydro-Electric Power Development." By G. V. Eckenfelder, M.E.I.C., presented to the 66th Annual Meeting of The Engineering Institute of Canada, at Vancouver, B.C. May 1952.

Fig. 9. View of Rundle power house and penstock

Fig. 8. Limberg dam and power house, looking upstream

The Glockner-Kaprun Scheme

P. M. Palmer outlines the main features of this important Austrian development and describes the progress of construction at the time of his visit in the autumn of 1953. When completed in 1957 this development will have a capacity of 332 MW.

PART TWO

Main Head Plant

Limberg Dam. The main dimensions of this structure are as follows:—

Maximum height above foundations	...	120 m.
Foundation width at mid cross-section	...	39·5 m.
Crown width at mid cross-section	...	6·0 m.
Length along crown (approx.)	... ...	350 m.

Concreting began in September 1948 and was completed by September 1951, the total volume of concrete placed being 446,000 cu. m. Steel shuttering was used throughout construction and all the concrete was placed by overhead travelling cranes which brought it in skips from a central mixing and batching plant on the west slope of the valley. During construction a total of approximately 122,000 cu. m. of overburden and 165,000 cu. m. of rock were excavated. All the raw materials for the manufacture of the concrete were brought to the site by cableway, cement from Kaprun, and sand and aggregate from a crushing and screening plant at Mooserboden. This last aspect of the work is described more fully in another section of this article.

The spillway and intake levels of the Wasserfallboden reservoir, formed behind Limberg dam, are 1,672 m. and 1,590 m. above sea level respectively. The spillway is built out from the east side of the valley, just upstream of the east abutment. A tunnel carries the water from the spillway, behind the abutment and discharges it on the downstream side of the dam.

Pressure Tunnel. From Wasserfallboden the water is conveyed to a surge tank at Maiskogel in a pressure tunnel 7·065 km. long, lined with precast concrete segments and increasing in internal diameter

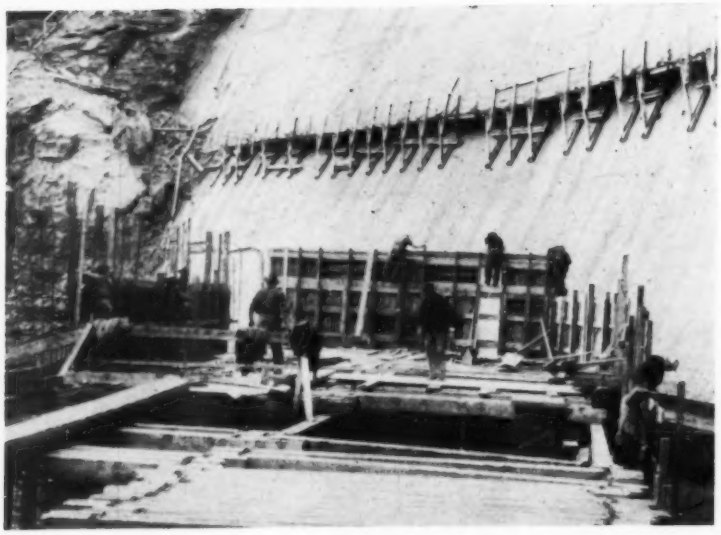


Fig. 9. Shuttering to roof blocks "A," "B" and "C" Limberg power house

from 3·20 m. at Wasserfallboden to 3·34 m. at Maiskogel. The tunnel has a fall of 2·6 per cent. and a maximum discharge of 32 cu. m. per sec. From the surge tank, which is excavated in the rock, a short length of tunnel, 576 m. long, takes the water to the control valves where three bifurcations guide it into four separate steel pressure pipes. This length of tunnel is lined with steel pipes, has an internal diameter of 3·0 m. and has been driven to a gradient of 18·3 per cent. Each of the four steel pressure pipes has an internal diameter of 1·40 m. at the control valves, reducing to 1·15 m. at their lower end. They are 1,200 m. long and have a wall thickness of 55 mm.

The pipes end in a single set of bifurcations just above the Kaprun power house, and penstocks carry the water to the turbines, in this case Pelton wheels. The Pelton-wheel nozzles are at a level of 781 m. above sea level and the main plant therefore operates under a maximum head of 811 m. This head is somewhat reduced as the level of the Wasserfallboden reservoir is drawn down during the winter months.

There are two openings in the pressure tunnel between Wasserfallboden and Maiskogel, at the Zeferet and Grub Valleys where the respective streams carrying the run-off from these areas are diverted into the water supply to the main plant at Kaprun.

Kaprun Power House. The Kaprun power house is situated at the foot of Maiskogel, on the outskirts of Kaprun and houses four main machines. Two of these comprise each a twin-runner twin-jet Pelton wheel turbine of 64,000 h.p., running at 500 r.p.m. and coupled to a 50,000 kVA generator producing 45,000 kW at 10 kV. The other two sets each consist of a twin-runner four-jet turbine of 83,000 h.p., running at 500 r.p.m. and coupled to a 70,000 kVA generator producing 55,000 kW at 10 kV. In addition, there are the two house generators each of 330

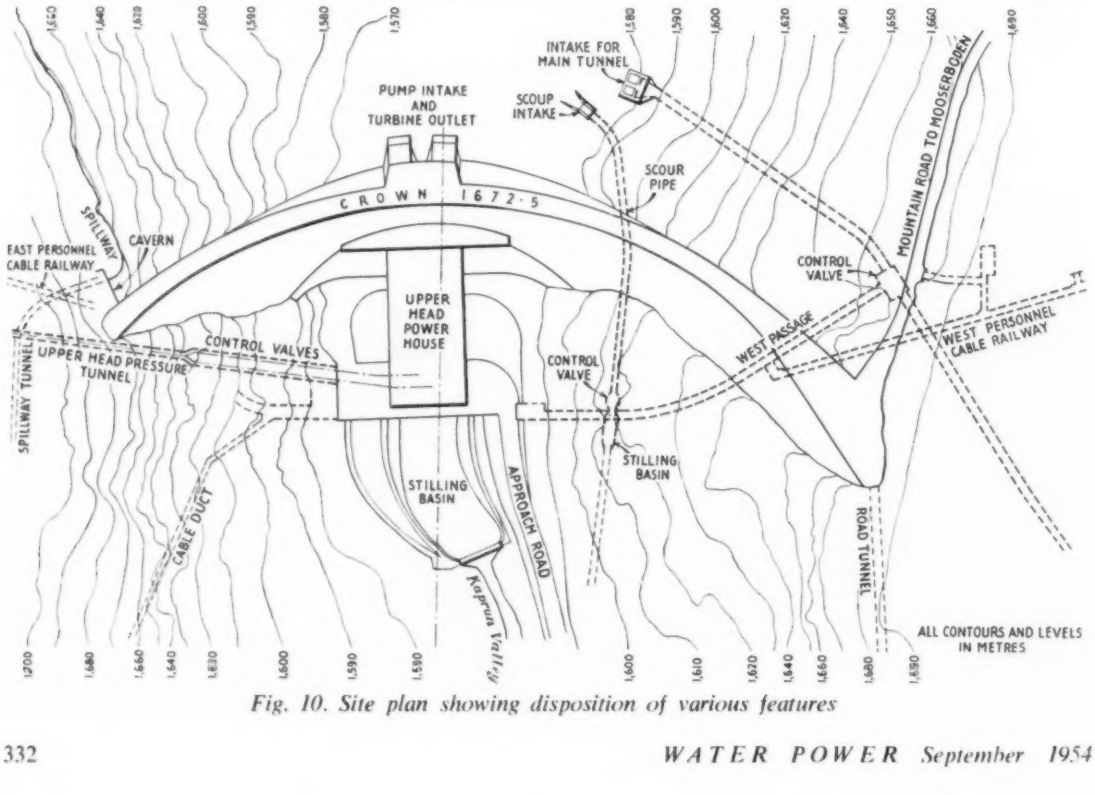


Fig. 10. Site plan showing disposition of various features

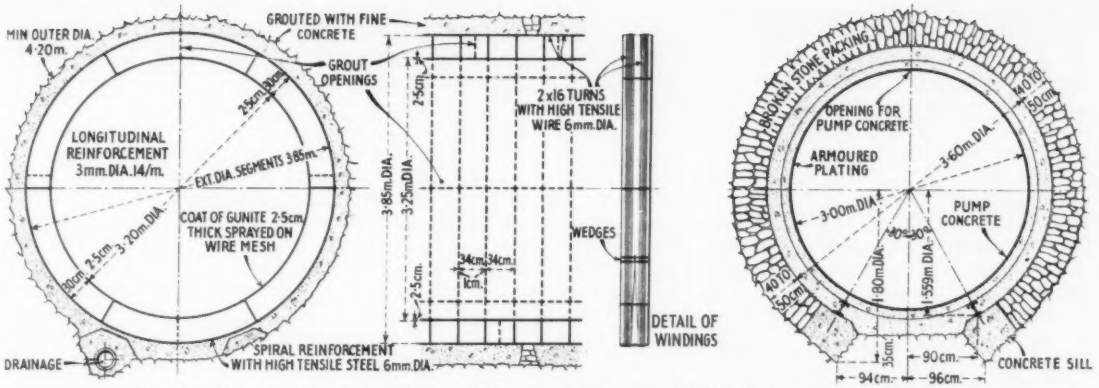


Fig. 11. Cross sections of Mooserboden-Limberg tunnel showing method of lining

kVA capacity producing 270 kW at 380 V, and each driven by a 365 h.p. Francis turbine running at 1,000 r.p.m.

The generators are connected to four transformers, two of 50,000 kVA and two of 70,000 kVA capacity and all four stepping up the voltage from 10 to 110 kV. The two house generators are connected to two 500 kVA transformers, stepping up the voltage from 380 to 16,000 V.

The turbines were fabricated and installed by Escher Wyss Ltd., and the electrical installations are by the Elin company*.

Transport and Communications

Because of the nature of the terrain in which the scheme is situated the transport of personnel and materials has from the outset been a major problem. The main railway runs as far as the power house at Kaprun and it is here that the main stores, repair shops, testing units, etc., are situated. From Kaprun a motor road runs as far as Kesselfell, a distance of 7 km., and this road has been extended a further 1·6 km. to the valley station of a funicular railway which climbs up to Limberg. From Kaprun to Kesselfell the road is 5·5 m. wide and climbs up a total height of 254 m. The last stretch of road from Kesselfell to the funicular railway, known as the Lärchwand road, is only 3·5 m. wide and traffic can only pass in one direction at a time. On both stretches of road the maximum gradient is 12 per cent.

At Lärchwand the road terminates at the valley station of the Limberg-Lärchwand funicular railway. This comprises a 8·1 m. gauge rail track, having a maximum gradient of 86 per cent, and a horizontal platform which is hauled up the track by two cables paid out from a winding house at the top of the railway. Each cable has a separate winding drum powered by a 250 h.p. motor, and as the cable is paid out it passes over pulleys set in the middle of the track at intervals along its whole length. The track is 812 m. long and the difference in elevation between the mountain and valley stations is 431 m. The platform has a maximum capacity of 60 tons, and with this load travels at 0·39 m. per sec. With a load of only 10 tons its speed is increased to 0·78 m. per sec. At these speeds the times taken for the whole journey from the valley station to the mountain station are 36 min. and 18 min. respectively.

From the mountain station a motor road, the Mooserboden road, runs via the crown of the Limberg dam up to the site at Mooserboden. Just below the mountain station is the "Middle Station" and from here a short length of road has been constructed to the Limberg power house.

The road from the mountain station to Mooserboden passes over the crown of Limberg dam and then climbs round the west side of the Wasserfallboden reservoir. The total length is 5·5 km. and because of the frequency of avalanches during winter 1,700 m. of this total length has been built in tunnels. The road is 3·5 m. wide and therefore restricted to one-way traffic, but passing bays have been constructed at intervals along its length. From the mountain station to Mooserboden the road climbs 400 m., the maximum gradient being 12 per cent.

There is another funicular railway at Maiskogel, connecting Kaprun power house with the surge tank at the head of the pressure pipeline. This funicular is 1,450 m. long, has a difference in level of 755 m. and has a maximum capacity of 18 tons.

During winter the Lärchwand motor road above Kesselfall and the Lärchwand funicular are closed and the only transport to Limberg is a small funicular railway which operates in the cable tunnel from Limberg power house to Ebenwald, just down the valley from Kesselfall. The maximum capacity of this funicular is only 5 tons; it is 923 m. long and has a difference in level of 590 m. A small 2 ton personnel funicular has been built in the west abutment of the Limberg dam to communicate between Limberg power house and the Mooserboden road over the crown of the dam.

Cement

Cement is brought from the manufacturers in special railway wagons as far as Kaprun, where it is discharged into storage silos. These silos have a total capacity of 1,000 cu. m. and cement can be discharged into them at a rate of 120 tons per hour.

From the silos at Kaprun the cement is conveyed by cableway in steel skips to the silos at Limberg, immediately over the crown of the dam on the west side of the valley. The total length of this cableway is 7·4 km. and the maximum capacity 35 tons per hour. At Limberg some cement is discharged into the silos and the remainder sent on a second cable-

* See "The Glockner-Kaprun Generators." By Prof. Dr. H. Sequenz. WATER POWER, November 1952, page 430.

way up to other storage silos at Mooserboden. The silos at Limberg are arranged so that skips come off the Kaprun cableway at one end of the building, are trundled along the length of the building by hand (the skips running on overhead rails) and are pushed on to the Mooserboden cable at the other end. When cement is required at Limberg the skips are diverted on to a side track and their load of cement discharged into the silos.

The cableway which carries the cement up to Mooserboden from Limberg is 3·5 km. long and has a maximum capacity of 30 tons per hour. On the Kaprun cable the skips travel at 2 m. per sec. and on the Mooserboden cable at 2·2 m. per sec.

The towers carrying the cables are fabricated in steel where they stand in the probable path of avalanches, otherwise they are always of timber. During the winter when no building work is in progress the cables are lowered to the ground to prevent their being damaged by avalanches.

Cableways are also used extensively for the transportation of personnel, and altogether four separate passenger-carrying cableways are in operation on the site. They are situated as follows: (1) between Kesselfell and Zeferet, at present carrying men and materials up to a construction camp at Zeferet but ultimately intended to provide access to the pressure tunnel where the Zeferet stream is diverted into it; (2) from Ebmatlen, on the Mooserboden road, across the valley to Höhenburg; (3) between Heilingenblut and Margaritze; and (4) along the Käfer valley to provide access to the Möll tunnel at kilometre 5·5.

In addition to these, two parallel cableways were used to transport sand and aggregate from Mooserboden to Limberg during the construction of the Limberg dam.

Construction in Progress

Mooserboden. Concreting in the Mooser dam was commenced in the summer of 1952 and since that date the Drossen dam also has been begun, concrete now being placed in both dams simultaneously.

Concrete is produced at a central batching and mixing plant, which, together with all the storage silos, is located on the Höhenburg. Sand and aggregate are obtained from natural deposits in the valley upstream of the dam and are taken by Krupp-Cyklop wagons and dumpers to a crushing, screening and washing plant on the east side of the valley. Here the aggregate is separated into six grades, 0·09—0·8 mm., 0·8—3 mm., 3—10 mm., 10—30 m.m., 30—80 mm. and 80—120 mm. All material larger than 120 mm. goes through a crushing mill before passing on to the screening plant. The aggregate is washed by spray jets as it passes along conveyor belts to the storage silos situated just below the screening plant. From here the sand and aggregate are taken in cableway skips to another set of silos on the Höhenburg. The combined capacity of the aggregate silos is 16,000 cu. m. and of the cement silo 3,500 tons, sufficient for four days' concreting.

The batching and mixing plant was fabricated by the Johnson Company of U.S.A. It has a maximum output of 4,000 cu. m. of concrete per day. At the top of the plant are seven storage hoppers, one for cement and the others for the six grades of aggregate, to which the cement and aggregate are brought by conveyor belt from the storage silos. From these storage hoppers the concrete mixes are batched in a weighing hopper and a carefully controlled amount of water added. Four tilting drum mixers stand beneath the weigh-batching plant and are each charged in turn. After the concrete is mixed it is discharged into dumpers waiting below. The whole operation, from when the material enters the weigh-batching plant until it is discharged as mixed concrete, is controlled by one man at a control desk on the weigh-batch platform.

Dumpers take the concrete to a loading platform where it is tipped into waiting skips, which are picked up by the overhead travelling cranes and taken to the

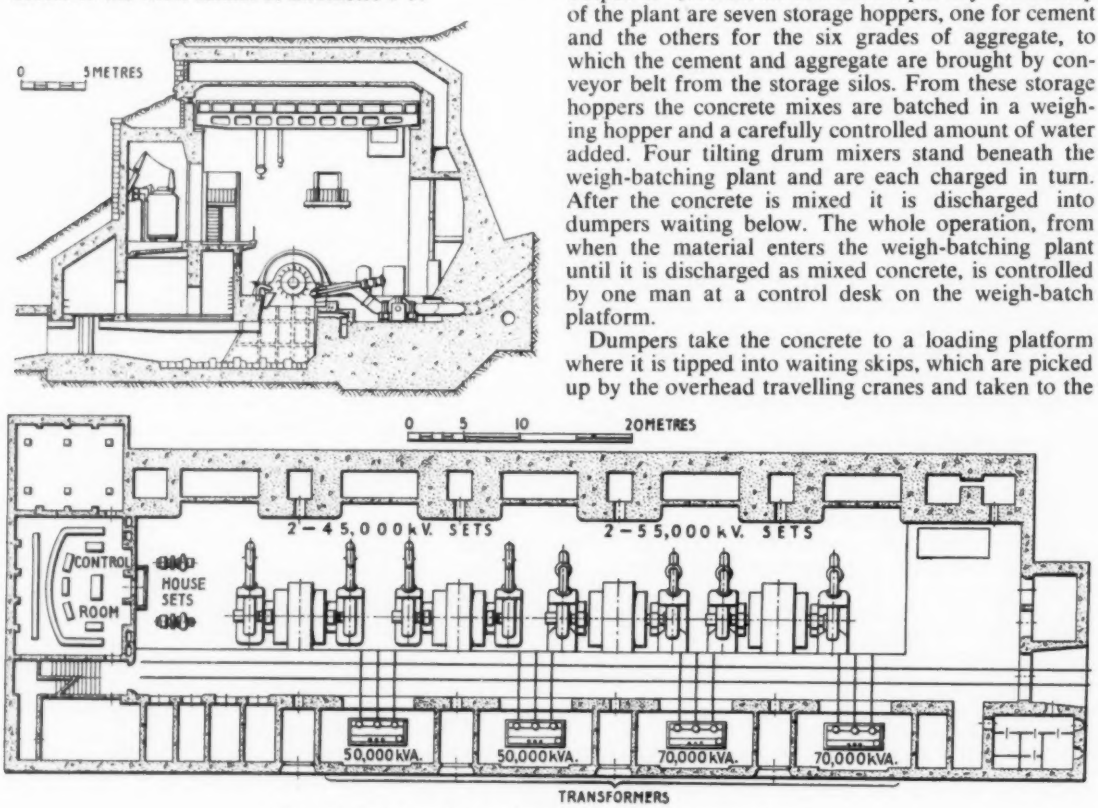


Fig. 12. Sectional elevation and plan of power house

placing position on the dam. There are altogether six cable cranes for placing concrete, three spanning either gorge, and as the winding houses are on the opposite sides of valleys to Höhenburg the crane drivers are directed by banksmen using telephone communication.

The skips are of the bottom-discharge type and the concrete is placed with heavy poker vibrators. Steel shuttering is used on both dams and concreting proceeds on alternate blocks. Vertical expansion joints are used between adjacent blocks, copper joints between upstream blocks and sheet-steel joints between downstream blocks. Eventually all joints will be grouted up.

To ensure a thorough bond between successive lifts the old surface is thoroughly scabbled, then hosed down with an air-water jet and a 5 cm. thick layer of grout applied before commencing placing the new lift. All concrete is cured for 14 days; vertical surfaces are kept damp by continuous spraying and horizontal surfaces by covering with damp sacking.

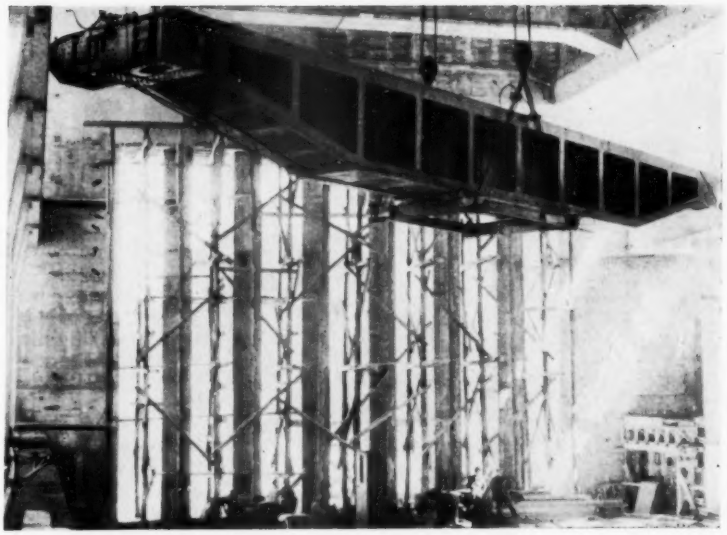


Fig. 13. Main crane beam being hoisted into position, Limberg power house

Limberg Power House. This is a reinforced-concrete structure built monolithic with Limberg dam. The work is on a much smaller scale than that at Mooserboden but has many interesting features.

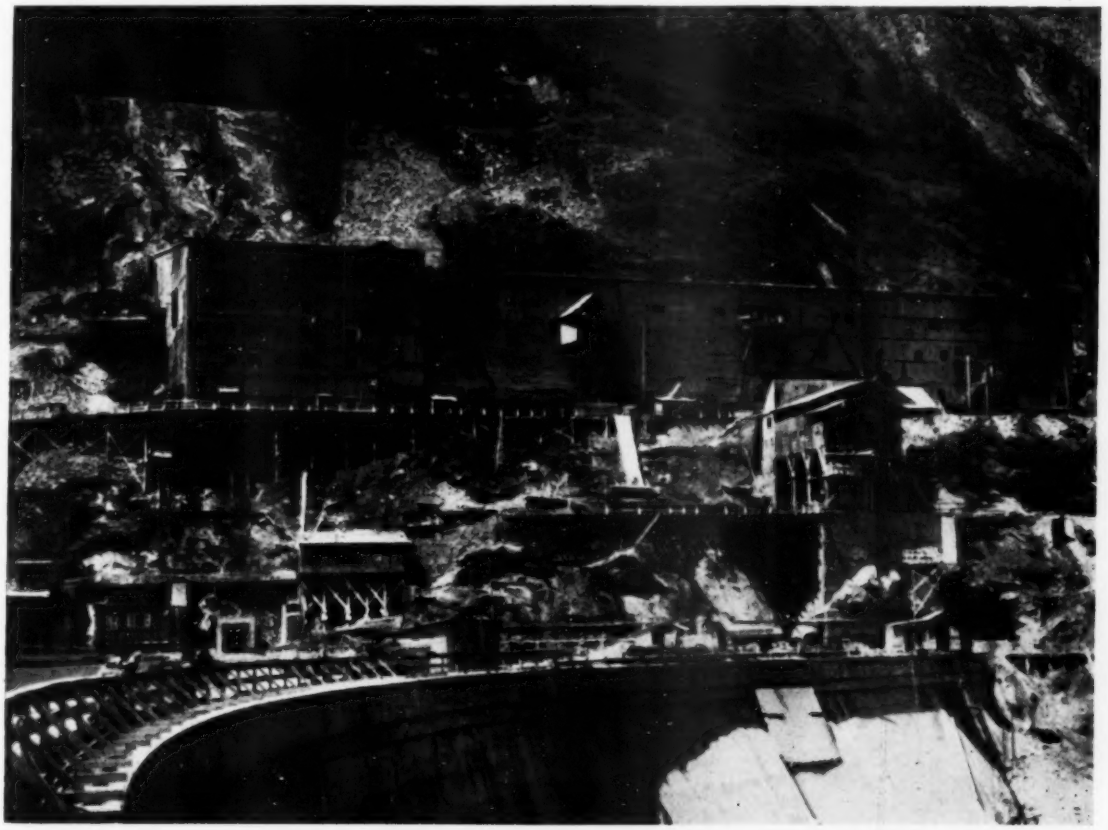


Fig. 14. Cement cableway station and cement silos from crown of Limberg dam

Fig. 15. Cement cableway from Limberg, looking towards Mooserboden

In contrast to Mooserboden all the shuttering on this site is of timber, chiefly because Austria has plentiful natural supplies of this material and it is therefore relatively cheap. The use of timber shuttering allows more scope in the design of concrete members but it does limit the height of the lifts in which concrete can be placed in walls to between 1 and 1½ m.

Sand and aggregate for the concrete are brought down from Mooserboden in Krupp-Cyklop wagons and stored in stockpiles against the downstream face of the Limberg dam. Cement is supplied from the Limberg silos through a 4 in. diameter pipe. A small batching plant stands on the west slope of the valley immediately beneath the aggregate stockpiles, and this serves two rotating-drum mixers.

All concrete is pumped into position by two piston-type pumps, each with a maximum capacity of 15 cu. m. per hour, through steel pipes approximately 6 in. in diameter, assembled from 6 ft. sections. To enable the concrete to be pumped satisfactorily a water/concrete ratio of 0·7 is used and Plastocrete added to each mix to improve the flow properties of the concrete.

The quality of the concrete is controlled by slump tests and compression tests. For the compression tests cubes are cast in steel moulds and tested after 19 days and 26 days curing.

Fig. 16. Cable railway approaching mountain station

Acknowledgments

In concluding this article the author wishes to acknowledge the use made of an article by Dr. Ing. J. Götz in the fifth issue (Jan. 1953) of the *Journal Tauernkraftwerk Aktiengesellschaft*, Zellam See. He is also indebted to the Chief Engineer of the Mayreder-Stuag, civil-engineering contractors, who gave much of his valuable time in showing the author over the various structures described.

Book Reviews

Hydraulics Research 1953. Issued by the Department of Scientific & Industrial Research and published by H.M. Stationery Office, York House, Kingsway, London, W.C.2. Price 7s. 8d., post free ($1·70).

The report of the Hydraulics Research Board and the report of the Director of Hydraulics Research, which are included in this publication, describe the progress which is being made in the building of the Hydraulics Research Station at Wallingford and the research work which has been carried on in 1953.

The first stage of the construction of the main hall has advanced during the year, and a second wave basin, 120 ft. by 80 ft. by 2 ft. 6 in., came into use during the summer. It is equipped with a large reversible circulating pump with a discharge of up to 74 cusecs for the development of tidal currents in coastal models. One of the models which will be constructed in the main hall will be a large-scale representation of the Severn estuary, on which the effects of the Severn Barrage on the estuary can be studied. A pilot model of the estuary constructed to scales of 1/8500 : 1/200 is being used for preliminary investigations to determine a satisfactory seaward limit for the large model.

Further work is connected with the problems created by the East Coast floods of 1953, and to this end survey parties from the station inspected damaged sea defences and beaches following the severe storm and tidal surges. Of more interest to the hydro-electric engineer is a sensitive current meter which has been devised for measuring a wide range of water velocities in models. A small propeller seemed to be suitable for the purpose, provided that a means of timing could be devised which did not interfere with its rotation. This was achieved by supporting the plastic propeller on a stainless steel rod which forms one electrode of an electrical circuit. The second electrode is provided by a gold wire mounted radially to, and just clear of, the propeller. Under these conditions the movement of a foreign body (in this case each propeller blade) across the conducting face of the gold electrode causes a change in the conductance between the two electrodes. Two separate timing devices have been developed to use this principle. One is a dekatron counter and timer, the other an electronic tachometer. By these methods it is possible to measure water velocities as low as one quarter of an inch a second.

Norske Kraftverker (Norwegian Power Plants). Published by *Teknisk Ukeblad* (Journal of the Norwegian Society of Civil Engineers), Kronprinzensgata 17, Oslo. Edited by Arne Solem assisted by Ragnar Heggstad and Nils Raabe. 1954. 12 x 8½ x 1⅛ in. 440 pp. Many illustrations. Price 125 Norwegian kroner, postage 6 kroner. Copies are on a numbered and named circulation, and are obtainable only on direct application. Printed in Norwegian.

Norway has by far the highest hydro-electric potentiality of any country in Europe, and development since the war has been phenomenal. In the light of this the decision of the Norwegian Society of Civil Engineers to celebrate its centenary by publishing a book on Norwegian water power is a particularly happy one, and the manner in which this decision has been implemented reflects the utmost credit both on the editor and his colleagues and on the Institution they serve. Norway has always taken a pioneer position in the design of hydro-electric schemes, and in this jubilee publication we are presented with a clear and comprehensive picture of contemporary Norwegian development and practice.

In his preface Mr. Solem acknowledges the indebtedness of the Institution for financial and other assistance from Norges Vassdrags-og Elektrisitetsvesen (Norwegian Watercourse and Electricity Board), and the book commences very fittingly with a general survey of Norwegian water-power development by Fredrik Vogt, the General Director of the Board. This is followed by an account of the activities of Samkjøringen (Norwegian Joint Electricity Supply Organisation) by C. B. Blydt, the head of that organisation, and a third article, on the legal aspects of watercourse developments, is contributed by Supreme Court Advocate R. Karlsrud.

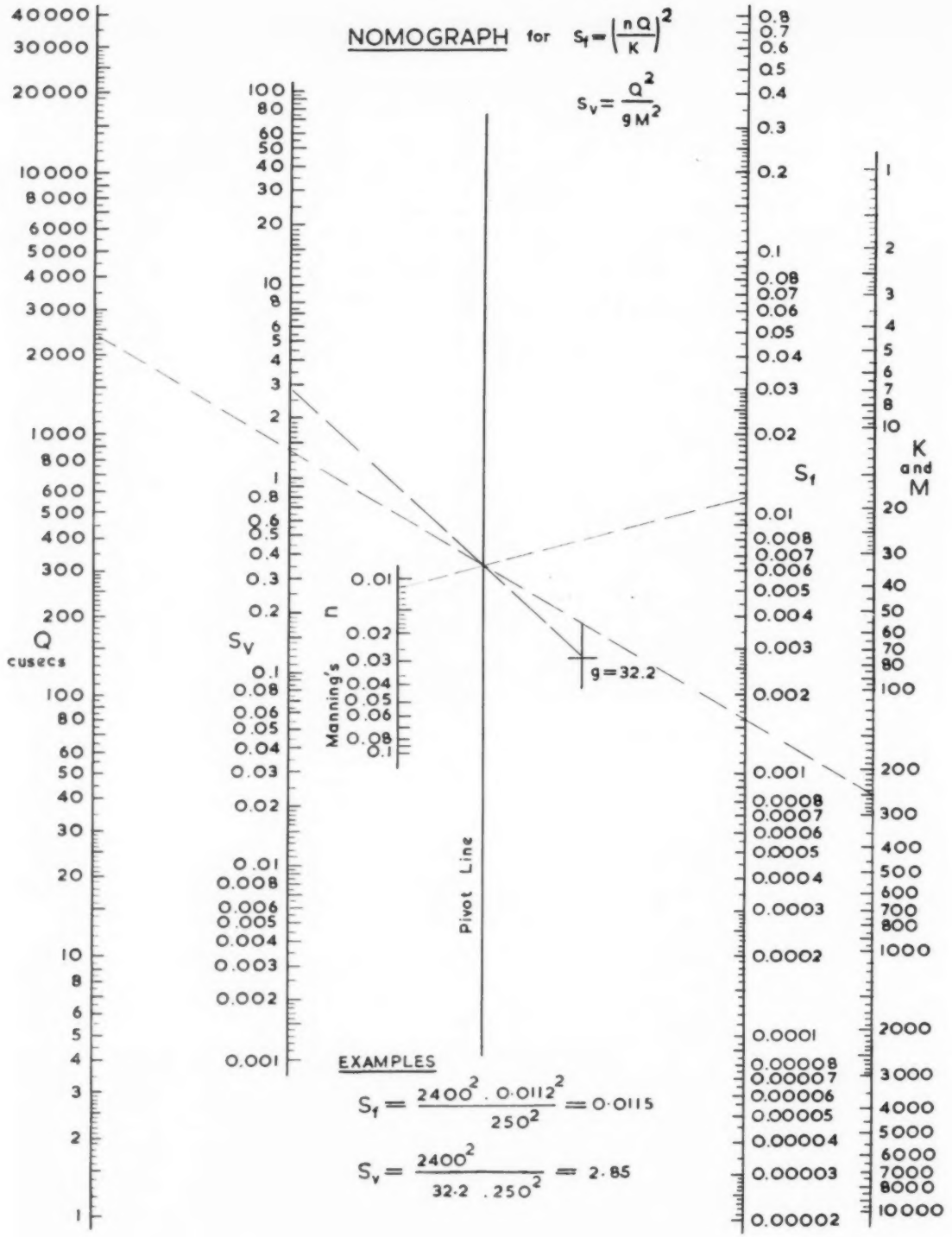
The bulk of the book, however, is occupied by descriptions of 190 hydro-electric plants. Even this large number is considerably less than one-tenth of the plants in operation in Norway, and it must have been no easy task to decide which plants to select for description. This task appears to us to have been accomplished very commendably, for the resulting conspectus is well balanced and gives due attention both to recent developments and to older stations of technical or historical importance. A great deal of trouble, too, has obviously been taken to reduce the mass of material received to a uniform and easily assimilable style of presentation. All descriptions are concise, but are replete with just those technical data that the engineering reader finds most useful.

The geographical locations and present and ultimate capacities of the plants described are conveniently collected in a table and an accompanying map, and there is also a map of Norway showing the average annual run-off.

Having regard to security regulations in Norway, the provision of illustrations is liberal, and the line drawings are particularly well executed. Indeed, the book is excellently printed and produced, and is handsomely bound.

I.S.M.E.S. (Experimental Institute for Models and Structures). We have received the first issue in English of the Bulletin of this new Italian Institute, which has been founded at Bergamo by a group of leading industrial and electricity concerns, in co-operation with firms of contractors, for carrying out research work and tests in the following fields: materials, models, structures and their elements, and measuring instruments. The Bulletin gives details of the organisation of the Institute, and describes the buildings together with up-to-date equipment of the various departments. A large proportion of the work done by the Institute is devoted to dams and other hydraulic structures.

Ball Valves. A coloured folder issued by S. Morgan Smith & Company, of Pennsylvania, U.S.A., illustrates and describes the various features of their ball valves. Dimensional data are given, with the names of the component parts designated in sectional drawings.



NEW GRAPHS AID OPEN-CHANNEL CALCULATIONS

Fig. 1. Nomograph for the solution of the general equation for flow in open channels

New Graphs Aid Open-Channel Calculations

R. Silvester, B.A., B.E., A.M.I.E.Aust., of the University of Western Australia, presents a series of graphs to simplify open-channel calculations.

THIS article illustrates the use of new graphs and a nomograph in the solution of the general equation

$$\frac{dy}{dx} = \frac{S_o - \left(\frac{Qn}{K}\right)^2}{1 - \frac{Q^2}{gM^2}}$$

for flow in open channels. Graphs are supplied for rectangular, circular, parabolic and trapezoidal channels in a large variety of dimensions. A knowledge of open-channel phenomena and water-surface profiling by the reader is assumed.

LIST OF SYMBOLS

A Area of water cross-section (ft.^{2}).
a constant used in formula for parabolic channel cross-section.
b_w water surface width (ft.).
dx relatively small distances measured along channel (ft.).
dy increments or decrements in depth (ft.).
g acceleration due to gravity (32·2 ft./sec.^{2}).
K factor $= 1{\cdot}486 \frac{A^{\frac{5}{3}}}{p^{\frac{2}{3}}}$
M factor $= \sqrt{A^3/b_w}$
M_c critical value of $M = Q/\sqrt{g}$
$M1$, $M2$, etc. numbers for water-surface profiles—see textbooks.
n Manning's friction coefficient (dimensionless).
p wetted perimeter (ft.).
Q discharge (cusecs).
R hydraulic radius (ft.).
S water surface slope (dimensionless).
S_f friction slope $= \left(\frac{Qn}{K}\right)^2$ dimensionless.
S_o bed slope (dimensionless).
S_v velocity component $= \frac{Q^2}{gM^2}$ (dimensionless).
v mean velocity of flow (ft./sec.).
y depth of cross-section (ft.).
y_c critical depth (ft.).
y_o normal depth (ft.).

General

In the following treatment *steady* flow only is considered—that is, flow in which the depth at each section of the channel remains constant with respect to time.

A useful formula to use for open-channel calculations and one that has an abundance of information available in regard to the friction factor is Manning's formula

$$v = \frac{1{\cdot}486}{n} R^{\frac{2}{3}} S_o^{\frac{1}{2}} \quad - - - - - - - \quad (1)$$

this can be expressed as

$$Q = \frac{1{\cdot}486}{n} A R^{\frac{2}{3}} S_o^{\frac{1}{2}}$$

or

$$\frac{Qn}{S_o^{\frac{1}{2}}} = 1{\cdot}486 \frac{A^{\frac{5}{3}}}{p^{\frac{2}{3}}}$$

The term on the right-hand side is a function of depth only and can be replaced by a factor K, so that

$$Q = \frac{K}{n}\sqrt{S_o} \quad - - - - - - - - \quad (2)$$

It has been the practice to combine the friction coefficient n with the factor K and use a term known as the "conveyance" of the channel, but by keeping them separate it is possible to graph K against depth y for any given cross-section of channel, regardless of friction. This has been done for various-shaped channels in Figs. 1 to 9 inclusive to a logarithmic scale.

Equation (2) is the general equation for *steady uniform flow* in an open channel, that is, flow when the mean velocity remains constant over the full length of the channel being considered, giving a constant depth in prismatic channels.

Steady non-uniform flow occurs in an open channel when the mean velocity varies over the length under study and is generally due to a variation in depth.

By considering the total head at any two consecutive sections a distance dx apart it can be determined that

$$S = \left(\frac{Qn}{K}\right)^2 + \frac{d}{dx}\left(\frac{v^2}{2g}\right) - - - - \quad (3)$$

i.e.

Surface Slope = friction gradient + velocity component.

This is based upon the assumption that the friction losses $\left(\frac{Qn}{K}\right)^2$ are the same for non-uniform flow as for uniform flow at the same depth. This is not beyond doubt, but the assumption is reasonable since only gradual changes in depth are involved.

The velocity component $\frac{d}{dx}\left(\frac{v^2}{2g}\right)$ for prismatic channels can be shown to be

$$\frac{Q^2 b_w \, dy}{g A^3 \, dx}$$

The factor A^3/b_w is a function of the depth y for any known cross-section and is termed the M function, such that $M = \sqrt{A^3/b_w}$.

Values of M have been plotted against y on the same graph as for K.

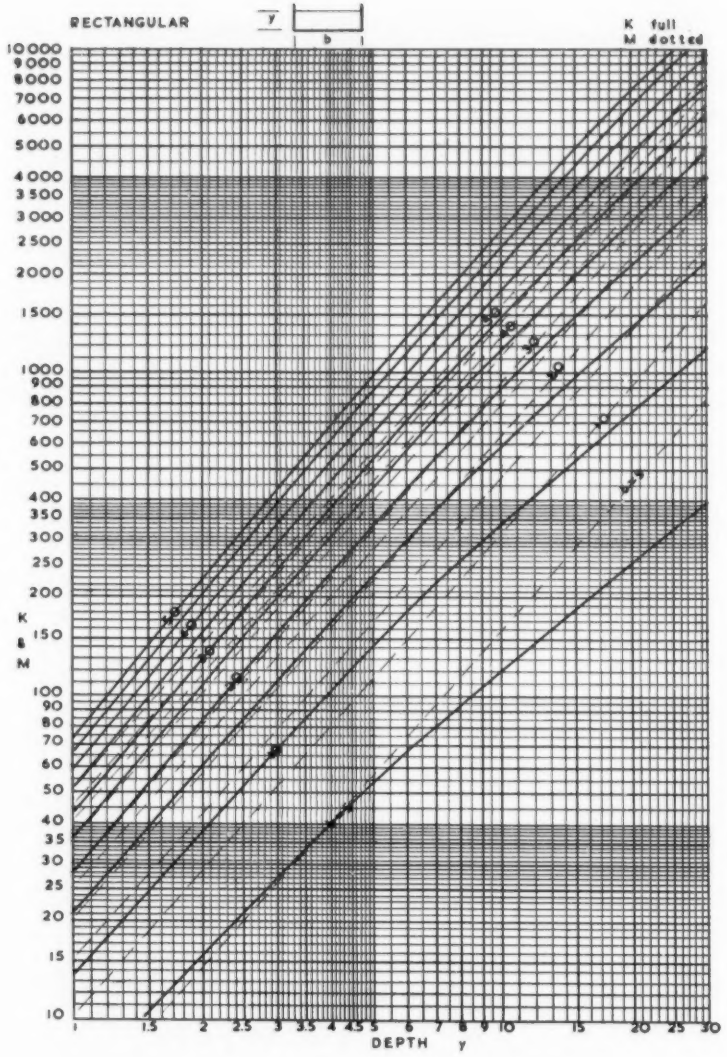


Fig. 2. Graph for K and M for rectangular channels

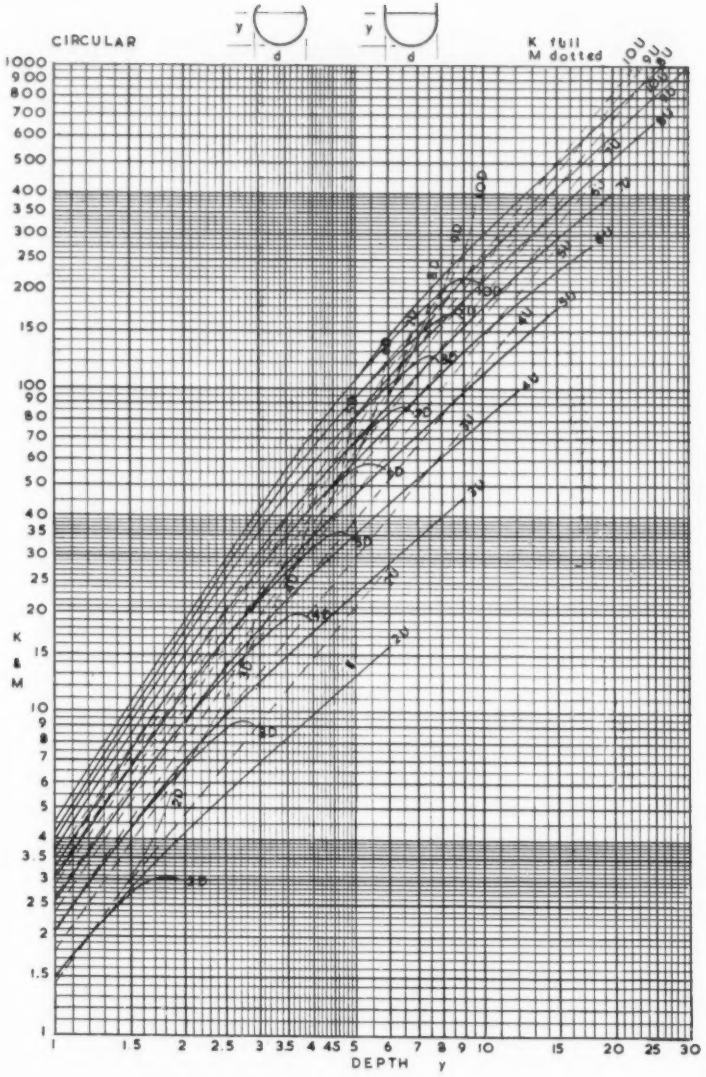


Fig. 3. Graph for K and M for circular channels

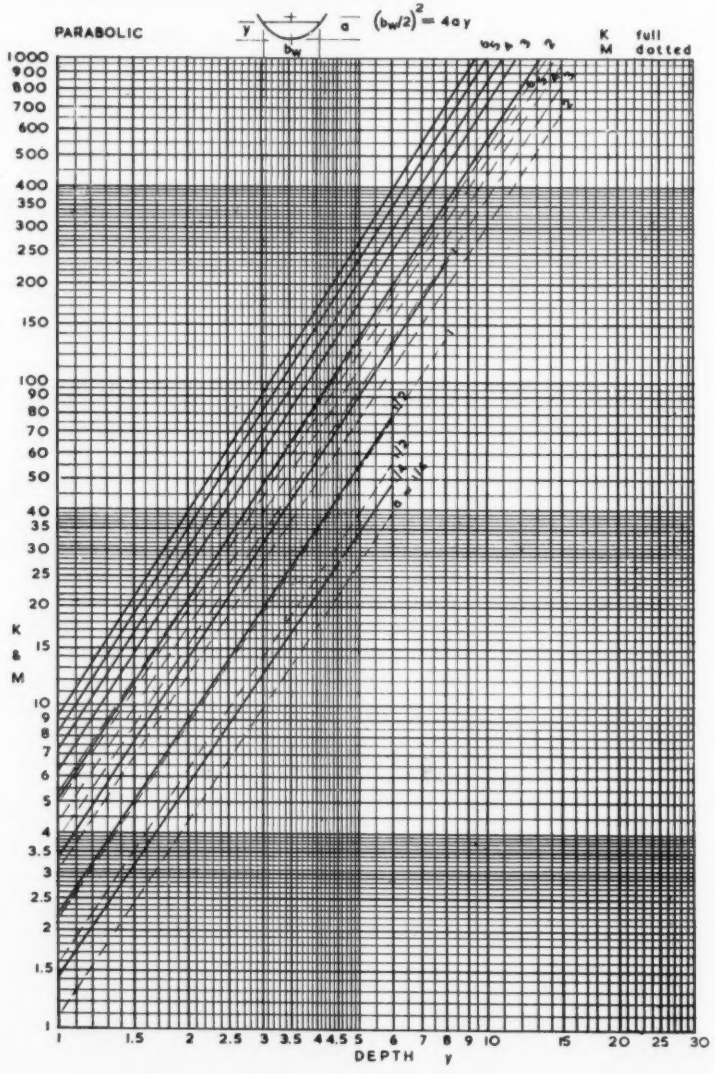


Fig. 4. Graph for K and M for parabolic channels

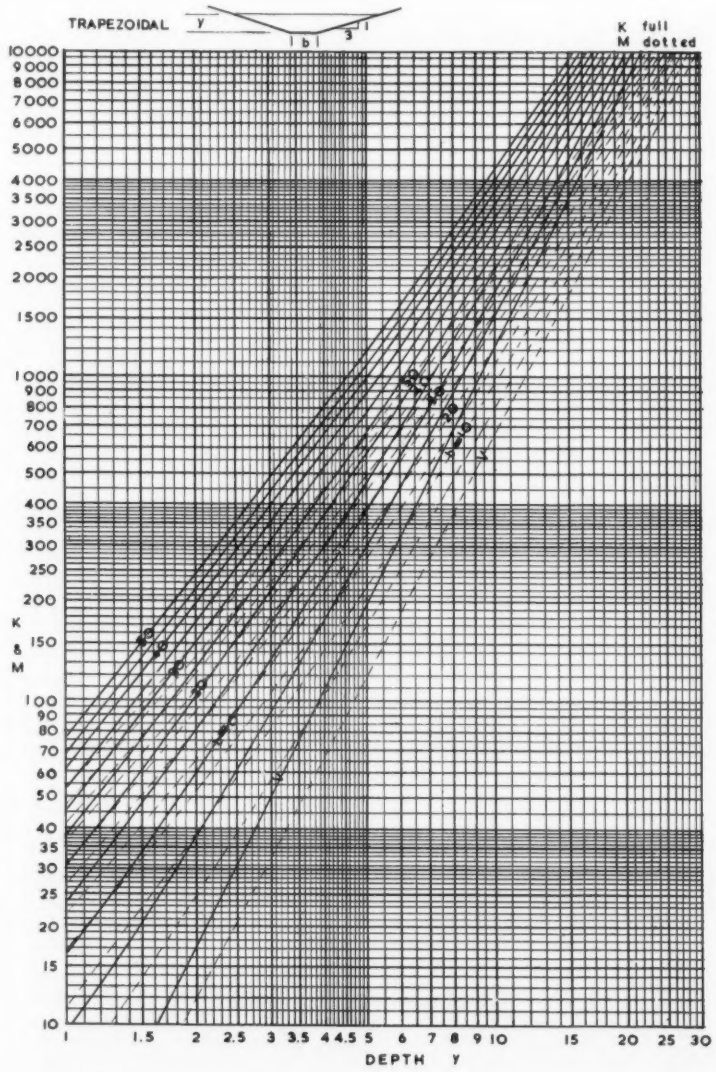


Fig. 5. Graph for K and M for 1 : 3 side-slope trapezoidal channels

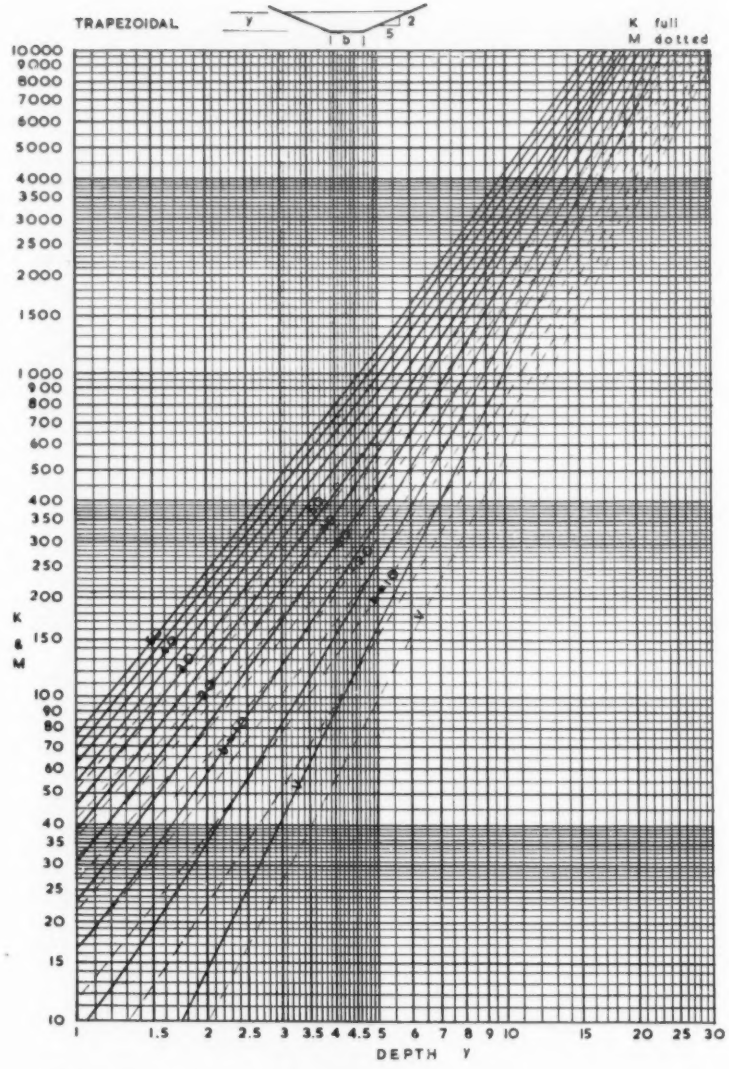


Fig. 6. Graph for K and M for 1 : 2·5 side-slope trapezoidal channels

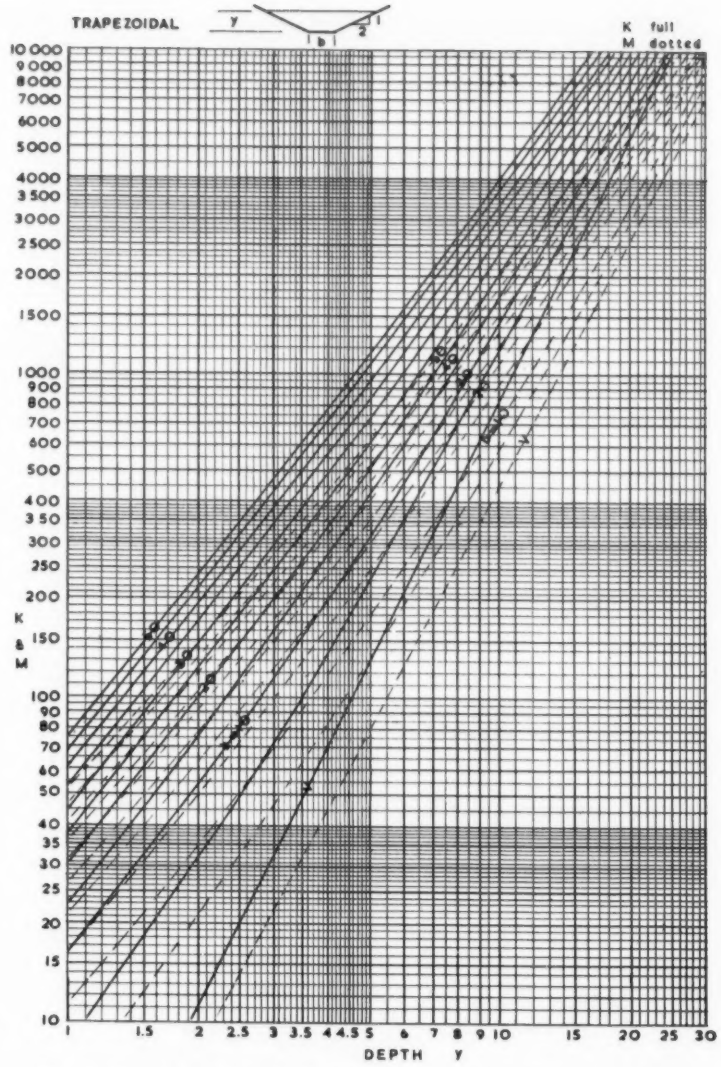


Fig. 7. Graph for K and M for 1 : 2 side-slope trapezoidal channels

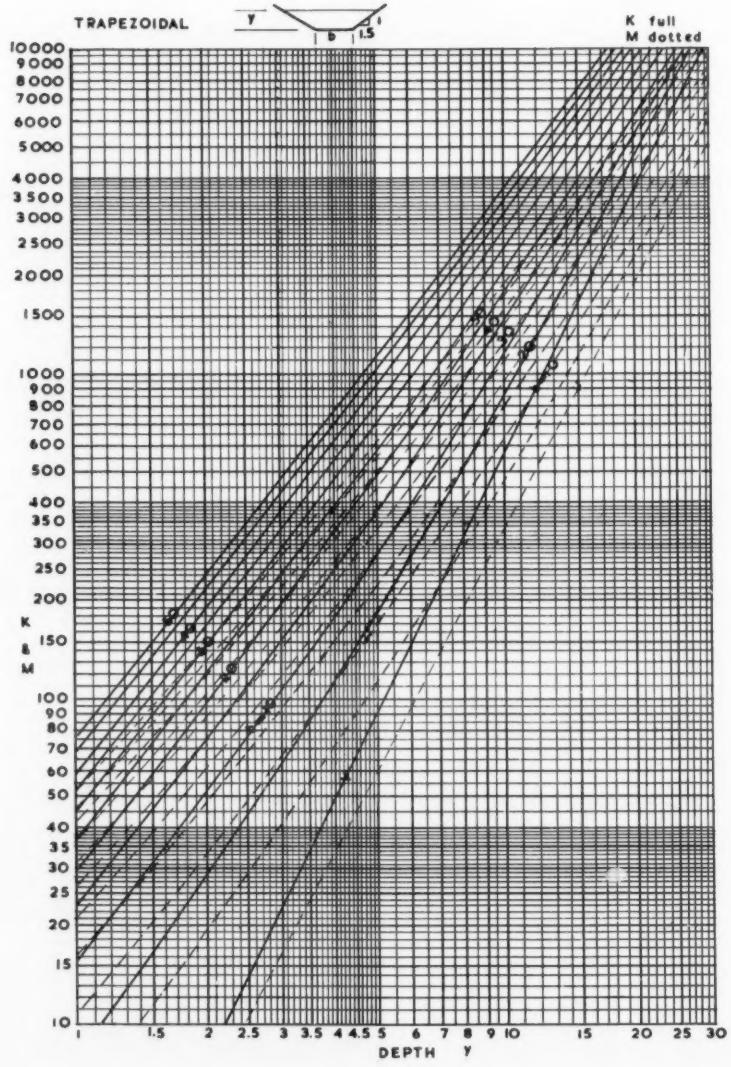


Fig. 8. Graph for K and M for 1 : 1·5 side-slope trapezoidal channels

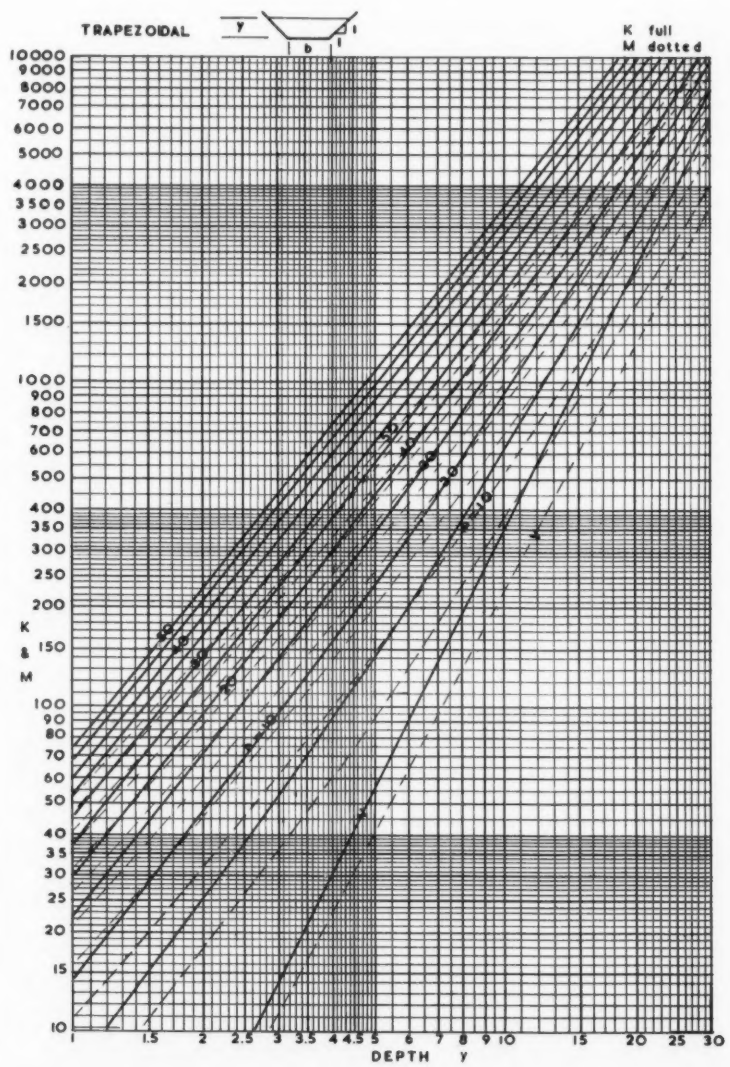


Fig. 9. Graph for K and M for 1 : 1 side-slope trapezoidal channels

Thus equation (3) becomes

$$S = \left(\frac{Qn}{K}\right)^2 - \frac{Q^2}{gM^2} \cdot \frac{dy}{dx}$$

and since $S = S_o - \frac{dy}{dx}$

we get $$\frac{dy}{dx} = \frac{S_o - \left(\frac{Qn}{K}\right)^2}{1 - \frac{Q^2}{gM^2}} \quad \text{- - - - - - - - - (4)}$$

or $$\frac{dy}{dx} = \frac{S_o - S_f}{1 - S_v} \quad \text{- - - - - - - - - (4a)}$$

It is seen in equation (4a) that when $S_o = S_f$ the numerator becomes zero and uniform flow results. Thus to solve equation (2) (the formula for uniform flow) the nomograph can be used for S_f and calling it S_o.

The remaining term $\frac{Q^2}{gM^2}$ can easily be evaluated from the nomograph.

Equation (4) is accurate for calculations of non-uniform flow in open channels as long as there is no substantial vertical curvature in the water surface, nor substantial divergence in the channel—such occurrences are called *local phenomena* and have a separate mathematical treatment.

Application of the Non-Uniform Flow Equation

Each term in the equation can be computed from the graphs and the nomograph and by tabulating them the value of $\frac{dy}{dx}$ can be determined quickly. The following procedure will minimise the possibility of errors:

(1) Determine the limits of the water surface at either end of the channel. Such limits are:
   (i) Uniform flow in long reaches of channel,
   (ii) A minimum depth downstream or at a fall,
   (iii) Computed depths
      (a) over a weir,
      (b) upstream or downstream of a sluice gate,
      (c) upstream or downstream of a hydraulic jump.
(2) From the general curves determine the profile that applies from the zone of flow and the bed slope.
(3) Choose depths at the extremities of the curves making sure that those in the vicinity of the normal depth y_o or the critical depth y_c are slightly greater or smaller than these values.
(4) Divide the total range in stage into about 10 increments dy and use the aids to solve equation (4) and so determine the lengths dx that give these increments.

A check should be made upon the sign of dy/dx as calculated and also upon its rate of increase or decrease, to see that it follows the curve type originally determined. This is necessary since any substitution will give a result the incorrectness of which may not be readily apparent.

The calculated values of dy/dx always err on the side of being smaller than the true variation in depth, therefore if decimals are to be eliminated when multiplying by dx, the larger values should be chosen. When the increments are specified, as suggested, and the *lengths* determined by dividing by dy/dx the *smaller* values should be preferred. Values of Manning's roughness coefficient are available in textbooks.

The following examples should illustrate the use of the graphs and nomograph.

EXAMPLE NO. 1

A parabolic channel whose cross-section is determined from the formula $(b_w/2)^2 = 4ay$ has water flow-

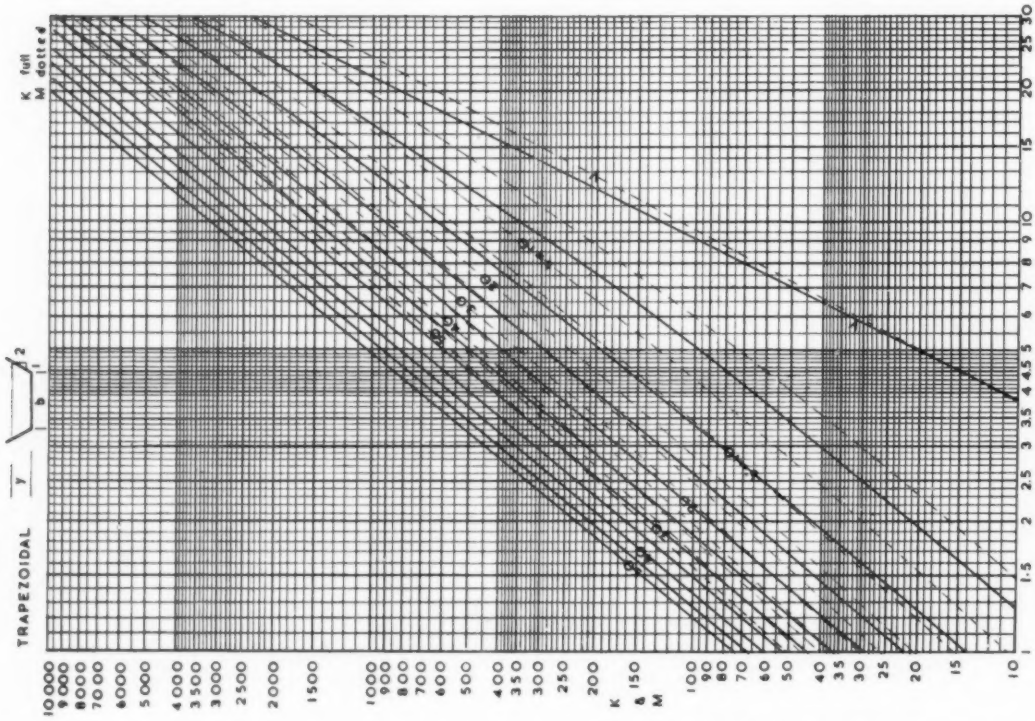


Fig. 10. Graph for K and M for 2 : 1 side-slope trapezoidal channels

ing in it at a uniform depth of 6 ft., the water surface width being 10 ft. If the bed slope is 0·0004, what is the discharge? Take Manning's n as 0·012. Is a hydraulic jump likely to form on this channel?

Since $(b_w/2)^2 = 4ay$, $\therefore (10/2)^2 = 4a6$, i.e. $a = 1{\cdot}04$ ft.

To determine K enter Fig. 3 at $y = 6{\cdot}0$ ft. and trace vertically to the K curve for $a = 1{\cdot}04$ (interpolated).

Thus $K = 130$.

To solve equation (2) use the nomograph as follows:

$n = 0{\cdot}012$ to $S_f = S_o = 0{\cdot}0004$.

$K = 130$ to $Q = 260$ cusecs.

To answer the second question it is necessary to find the critical depth.

From the nomograph for $S_v = \dfrac{Q^2}{gM^2}$ if S_v is unity then $M = \sqrt{\dfrac{Q}{g}}$ which is the critical value of the M function

thus $S_v = 1$ to $g = 32{\cdot}2$

$Q = 260$ to $M_c = 49$

Again entering Fig. 3 this time at $M_c = 49$, trace horizontally to the M curve for $a = 1{\cdot}04$ and the critical depth is found to be 5·5 ft.

Since $y > y_c$, and the slope is *mild* a hydraulic jump cannot form.

EXAMPLE NO. 2

It is desired to deliver 200 cusecs along a U-shaped channel that must have a bed slope of 0·0006. If the channel is to be constructed within a width of 6 ft. what major proportions of channel will suffice? Manning's $n = 0{\cdot}011$.

From the nomograph with $Q = 200$ cusecs

$S_o = 0{\cdot}0006$

$n = 0{\cdot}011$

we find $K = 86$

Entering Fig. 2 at $K = 86$ and tracing horizontally, it is seen that for width 4 ft. the depth is 10·7 ft.

" 5 ft. " " " 8·2 ft.

" 6 ft. " " " 6·6 ft.

For any widths between or beyond these the depths can be read directly. The topography and construction facilities will determine the best section to be adopted.

EXAMPLE NO. 3

A rectangular channel 30 ft. wide with a slope of 0·001 delivers 1,100 cusecs into a reservoir. The depth at the junction is 10 ft. It is required to find the distance upstream where the stage is within 6 in. of the normal depth. Manning's n may be taken as 0·02.

From the nomograph $K = 660$

thus $y_o = 6{\cdot}3$ ft.

From the nomograph $M_c = 200$

thus $y_c = 3{\cdot}7$ ft.

Since $y_o > y_c$ and $y > y_o$ the curve is of the form $M1$ (Fig. 11).

It is necessary to trace the depth variation from $y_1 = 6{\cdot}3 + 0{\cdot}5 = 6{\cdot}8$ to $y_2 = 10{\cdot}0$ ft.

The calculations are tabulated below.

y	dy	K	M	S_f	S_v	$S_o - S_f$	$+dy/dx$	dx
6·8	0·2	850	530	0·00068	0·15	0·00032	0·000376	530
7·0	0·3	900	550	0·00062	0·135	0·00038	0·000439	680
7·3	0·3	950	570	0·00055	0·125	0·00045	0·000514	580
7·6	0·4	1000	610	0·0005	0·113	0·0005	0·000564	700
8·0	0·5	1080	660	0·00042	0·092	0·00058	0·000639	780
8·5	0·5	1175	730	0·00035	0·076	0·00065	0·000704	710
9·0	0·5	1270	800	0·0003	0·065	0·0007	0·000748	660
9·5	0·5	1365	860	0·00026	0·056	0·00074	0·000784	630

Total 5270 ft.

Ans.: 1 mile

EXAMPLE NO. 4

A stream has such a cross-section that within reasonable accuracy it can be represented by a trapezoidal channel 50 ft. bottom width and side slopes 1 to 1. Over a straight stretch of half a mile five stations equidistant from each other were located where simultaneous recordings of water-surface levels were made. These readings, reduced to the value of 100 for the upstream water level, were 100, 99·19, 98·2, 96·91 and 95·06. The levels of the bed at the upstream and downstream end to the same datum were 86·0 and 84·68. The bed slope is uniform over the reach. What is the discharge of the stream taking Manning's roughness coefficient as 0·03?

From the sketch of the profile as indicated by the data supplied (Fig. 12a) it is apparent that the curve is type $M2$. This can be checked when the quantity is known.

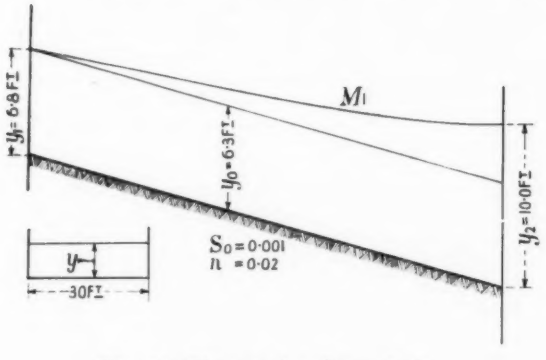


Fig. 11. Diagram for Example 3

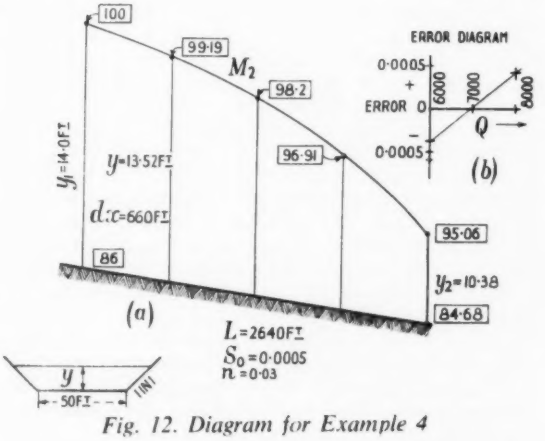


Fig. 12. Diagram for Example 4

Considering equation (4) the term dy/dx can be ascertained for the four stations, in this case by assuming straight-line water surfaces between each station since the distances 660 ft. are relatively short. In other circumstances it may be necessary to draw the profile to a large scale and draw intermediate chords to the surface. The depth at each station is known whence K and M can be found.

It only remains to test values of S_f and S_v for trial values of Q to satisfy equation (4a). By making two or three trials, aided by the nomograph, and by constructing an "error" diagram, the correct quantity to satisfy the equation can be arrived at quickly.

In the present problem one can make an intelligent guess at a quantity by considering 14 ft. (the upstream depth) as the normal depth and finding the discharge for uniform flow.

From the nomograph $Q = 4{,}600$ cusecs.

Since the curve appears to be an $M2$, it indicates that a greater discharge than for uniform flow exists, so that one may reasonably commence trials at 6,000 cusecs. It is realised, of course, that the normal depth to which line this water surface is asymptotic will be greater than 14 ft., the degree depending upon the section of the profile actually measured.

Below are tabulated the calculations for Station 1, where $y = 14{\cdot}0$, $K = 6{,}200$, $M = 3{,}100$ and

$dy/dx = -0{\cdot}00073$.

Note that when calculating dy/dx for any station it should be for the section of water surface nearest to the critical-depth line. This is because the chord approximates more closely the true slope of the water surface at the smoother end of the curve.

Trial Q	S_f	S_v	R.H.S. Equ. 4a	Error
6000	0·0008	0·13	−0·000345	−0·000385
8000	0·0014	0·23	−0·00117	+0·00044*
7000	0·0011	0·18	−0·00073	zero

* From the "error" diagram (Figure 12b) drawn at this stage, it is seen that 7000 cusecs may be the correct discharge.

This value of Q should then be checked at the other stations.

For $Q = 7{,}000$ cusecs, $y_o = 18{\cdot}0$ ft. and $y_c = 8{\cdot}0$ ft.

Thus the curve is type $M2$ since $y_o > y_c$ and $y_o > y > y_c$.

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Steady Flow in Open Channels: Woodward & Posey (John Wiley) 1941.

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"Dimensionless Constants for Hydraulic Elements of Open Channel Cross-Sections." P. Z. Kirpich. *Civil Engineering*, 1948, vol. 18, p. 47.

"Flow in Open Channels." R. Silvester. *Journal, Inst. Eng. Aust.* 1953, vol. 25, p. 155.

Film Library Cavalcade

By way of celebrating their 21st anniversary, G.B. Film Library, part of the J. Arthur Rank Organisation, have been showing a cavalcade of 16 mm. films at the National Film Theatre. In introducing the Industrial Session, Sir Harry Pilkington, President of the Federation of British Industries, stated that films could now be advantageously applied to many purposes in the industrial world, ranging from such internal uses as recording constructional progress in major civil-engineering works, training operatives and promoting understanding and good relations within a firm, to the outward-looking type which told a sales story to the public. In connection with the latter type, he referred to technical information films by means of which an overseas salesman could show a prospective buyer how a piece of machinery was constructed, how it was operated, and how it was serviced. In this country, he added, there were over twenty thousand 16 mm. projectors installed in schools and other educational institutions, and he considered this to be of the greatest importance, as these installations afforded boys an opportunity not only to appreciate the essential part that industry is playing in sustaining the well-being of the community, but also to think of industry in terms of their future careers. In his own organisation, he went on to say, films had been used for many years to demonstrate to clients how glass was made and used. Besides telling its own story a film of this kind often served the salesman as a useful means of introduction to people he wished to interest.

The G.B. Film Library was established 21 years ago, and not only has a vast selection of films which it is prepared to send to any part of the world, but also provides an advisory service to firms who wish to make their own films.

New Experience in Mass Concrete Construction

We regret that in the article which appeared under this title in our July and August issues, the following footnotes were omitted:—

[1] Fritsch: "Amerikanischer Talsperrenbau." Schriftenreihe des Osterreichischen Wasserwirtschaftsverbandes, Heft 24, Seite 10, Verlag Springer, Wien.

[2] Fritsch: "The Strength of Concrete," Civil Engineering and Public Works Review, London, June 1953.

[3] Fritsch: "Talsperrenbeton, Sicherheit und Verantwortung," Schriftenreihe des Osterreichischen Wasserwirtschaftsverbandes, Heft 15, Verlag Springer, Wien.

[5] Fritsch: "Vibrated Mass Concrete," Civil Engineering and Public Works Review, London, Jänner u. Februar 1951.

[6] Kolb: "Uber Betonverdichtung durch Innenrüttler," Die Bautechnik, 1949, Heft 2.

[7] Fritsch: "Der heutige Stand der Massenbetontechnik," Schriftenreihe des Osterreichischen Wasserwirtschaftsverbandes, Heft 19, Verlag Springer, Wien.

Concrete Pumps for Hire. A new company for the hire of concrete pumps has recently been formed. This is known as Pumpcret Hire Co. Ltd., and is an associate company of The Concrete Pump Co. Ltd., both of 4, Stafford Terrace, London, W.8. The new company will hire Pumpcret equipment, mainly in the United Kingdom.

Dick's Asbestos and Allied Products. A new catalogue issued by Dick's Asbestos & Insulating Co. Ltd. describes an extensive variety of packing, jointing and insulating materials manufactured by this firm, which is a subsidiary of Thos. W. Ward Limited. Of interest to hydro-electric engineers are hydraulic gland packings for medium and high pressures, conveyor belting and belting for mechanical drives.

The Gal Oya Development

One of Ceylon's major multiple-purpose developments is described by S. M. Alexander, Civil Engineer, International Engineering Company.

PART ONE

IN 1948 the Ceylonese government under the late Right Honourable D. S. Senanayake, its Prime Minister, initiated a six-year programme of accelerated economic development. A substantial portion of this effort was directed towards the development of unused lands in the Dry Zone of the island. The largest single step toward this objective has been the multiple-purpose Gal Oya project which will provide water for irrigation of 120,000 acres of land, approximately 86,000 acres of which are new lands to be reclaimed from jungle. Incidental benefits to be derived from this project are power generation and flood control.

A significant factor in effecting this development has been the utilisation by the Government of Ceylon of American engineers for a considerable portion of its design and a large American contractor using modern methods and equipment for its construction. The project was initiated by the Irrigation Department of the Ministry of Public Lands which supervised the construction of principal features. Operation of the project, including settlement of farmers on the new lands, is under the direction of the Gal Oya Development Board.

Purpose of the Gal Oya Project

Climatologically, the island of Ceylon is divided into two general areas, the Wet Zone, which is the south-west portion of the island and containing roughly a quarter of the island's 25,000 square miles, and the Dry Zone which includes the remainder of the island. Most of the rainfall on Ceylon occurs during the monsoons, the south-west from May to September and the north-east from November to March. Rainfall from the north-east monsoon is distributed generally over the entire island. A range of mountains in the south central part of the island acts as a barrier to the south-west monsoons which deliver the major part of their moisture to the Wet Zone. Rainfall in the Wet Zone is consequently both abundant (75 to 150 in. annually) and favourably distributed for agriculture. From a casual inspection of precipitation figures, the term "Dry Zone" may appear to be a misnomer. Annual rainfall in the Dry Zone is generally in the order of 50 to 75 in. with the exception of limited coastal areas in the west and south-east portions of the island where the figure is somewhat under 50 in. The distribution of this rainfall is, however, unfavourable to agricultural development since about 80 per cent. occurs during the north-east monsoon and inter-monsoon periods. The ancient farmers learned that storage of water during the north-east monsoon for subsequent use during the dry period was essential to successful agriculture.

Irrigation works have been constructed in the Dry Zone since ancient times. Many of these systems were quite extensive and complex, attesting to the highly advanced state of the early cultures of Ceylon. Reservoirs (termed "tanks" from the Portuguese tanque) were constructed by damming river valleys with earth dams (bunds) provided with masonry spillways. Another form of irrigation works consisted of overfall dams (anicuts) which diverted water into canal systems for use on remote fields. Most of these fell into ruin as invasions, wars, population migrations, and extreme floods took their toll. Restoration of

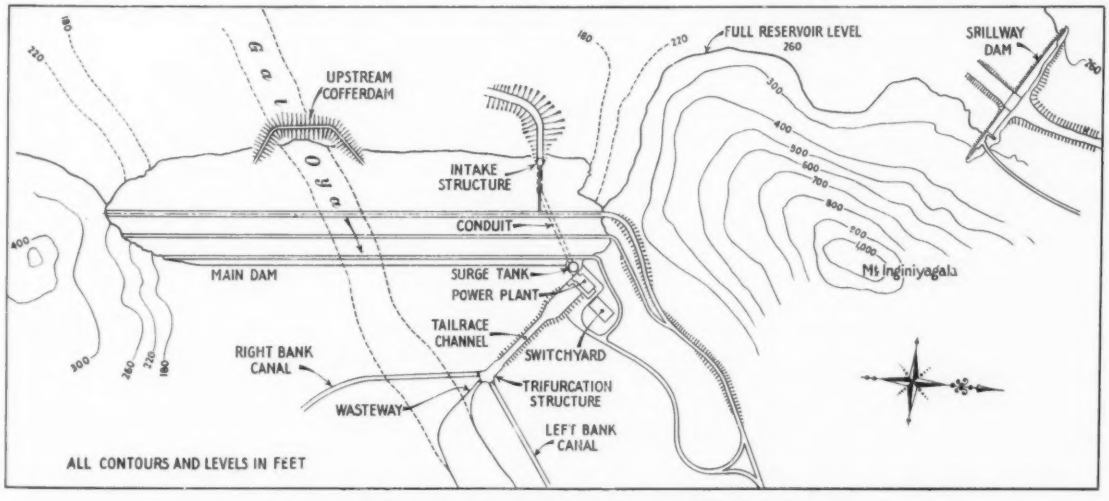


Fig. 1. Site plan showing main features of Gal Oya scheme

Fig. 2. Preparation of main dam foundation, showing dome-shaped rock

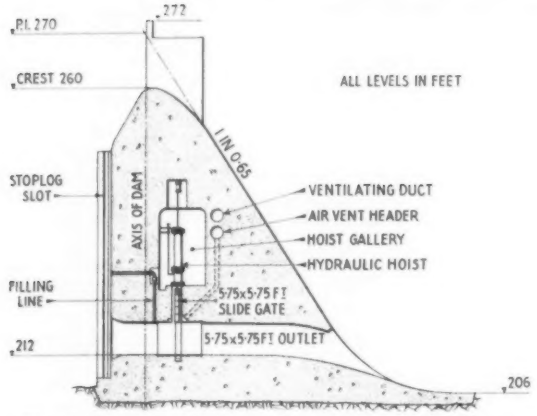


Fig. 3. Uncontrolled overflow section of spillway

these works was initiated by the Dutch in the course of their administration of the island in the eighteenth century and again during the latter part of the nineteenth century under British administration.

At present about 70 per cent. of the island's population of eight million (1953 estimate) is concentrated in the Wet Zone. Additional concentrations are present at the northern tip of the island and near the port of Batticaloa on the eastern coast, but the remainder of the island is sparsely settled. In developing lands within the Dry Zone, most of the effort of the Department of Irrigation has been directed towards rehabilitation and improvement of ancient irrigation works. Nearly 10,000 early works are known and more than half of these have been restored. By far the outstanding new development has been the Gal Oya project which is unique both in magnitude and in the method of implementation.

The Gal Oya (literally translated "Rock Creek") rises in the central hills and flows eastward 62 miles through the Dry Zone, discharging to the sea about 30 miles south of the east coast port of Batticaloa. It has a watershed of 700 square miles receiving 75 to 100 in. of rainfall annually. At the dam site, located 27 miles from the coast, the drainage area is 384 square miles which receives an average annual rainfall of 82 in., of which 62 in. occur during the north-east monsoon. The average annual runoff at the dam site is 740,000 acre-feet of which 650,000 acre-feet occur during the north-east monsoon. Records to 1947 revealed two flood peaks slightly in excess of 85,000 cusecs in observations over a period of nine years.

History of the Gal Oya Project

Preliminary surveys of the Gal Oya valley were initiated by the Irrigation Department in 1937 at which time the site at Inginiyagala Mountain was established. Detailed surveys were made at this site and completed in 1947. Dr. John L. Savage, famed as the former Chief Designing Engineer of the U.S. Bureau of Reclamation, was engaged as a consulting engineer by the Government of Ceylon in 1946 and upon his recommendation the project received the sanction of the Government in 1947. Shortly thereafter the Department of Irrigation entered into an agreement with International Engineering Company, Inc., of San Francisco, for the preparation of contract drawings and specifications for the Gal Oya dam and power plant. During this phase of the work, Mr. D. W. R. Kahawita was in the United States as representative of the Ceylonese Government. In 1948 the Irrigation Department entered into an agreement with Morrison-Knudsen International, Inc., to construct the Gal Oya dam and power plant. At the same time the original contract with International Engineering

Company was extended to include the preparation of detailed construction drawings. American construction personnel arrived at the site in February 1949, and all work under the original agreement, as well as several supplemental contracts, was completed in December 1952.

General Description

The Gal Oya project incorporates two dams: (1) the main dam, an earth embankment with a length of 3,600 ft. and a maximum height of 120 ft. above the original streambed; and (2) the spillway dam, a concrete structure with a length of 1,050 ft. and a maximum height of 84 ft. above the lowest point of the foundation. Inginiyagala Mountain, with its summit at elevation 1,085, separates the two dams and majestically dominates the scene. In addition, there are several minor earth dikes filling in low saddles in the reservoir rim. The reservoir has a length of 11 miles, a maximum width of 7·5 miles and a capacity of 770,000 acre-feet with the surface at the maximum storage elevation 260. Thus, the capacity is somewhat in excess of the average annual runoff. Two 2,750 kVA units are installed with provision for two additional units for an ultimate capacity of 11,000 kVA. Outlet works are provided to meet irrigation demands when these exceed power requirements. Main canals on both banks of the river distribute water to secondary systems downstream. Fig. 1 shows a general layout of the project.

Main Dam

Foundation. The foundation for the Gal Oya dam is a very hard granite gneiss overlain by weathered rock and alluvium. A cutoff trench was excavated through the surface material to bedrock under the central third of the dam. Exposure of the rock surface by the cutoff excavation revealed it to be very rough and uneven with dome-shaped protuberances over most of the area (Fig. 2). Original plans envisioned a concrete cutoff wall along the entire length of the dam, excavated 10 ft. into bedrock and extending 10 ft. into the fill. Without considerable excavation the profile of such a wall would have been very irregular and consequently costly. The functions of such a wall would have been: (1) to seal cracks and fissures where the surface of the rock is shattered, and (2) to key the impervious core to bedrock. Inasmuch as the rock exposed was exceptionally sound with few cracks or crevices, and the domed rock surface would serve amply to key the impervious core to the foundation, the cutoff wall was eliminated over most of the foundation. The cutoff wall was retained at both abutments, which are fairly steep. The entire rock surface was hand cleaned before placing embankment, and unsuitable material was removed from the remainder of the foundation area.

A grout curtain was formed along the entire length of the foundation by a line of holes 50 ft. deep at 10 ft. centres located at about the upstream third point of the cutoff trench (along the line of the intended cutoff wall). A total of about 20,600 linear ft. of $1\frac{3}{8}$ in. diameter holes were core drilled for this purpose. Two additional lines of holes, 20 ft. deep on 20 ft. centres, 20 and 40 ft. upstream of the main line were drilled as required. About 8,400 linear feet of these holes were percussion drilled. Many of the holes along the main grout line took little or no grout. In all, 5,785 cu. ft. of cement grout was required.

A 15 to 20 ft. wide fissure cutting across the foundation and filled with a tight claylike material was encountered. The material was removed to a depth of 20 to 25 ft. below the general rock level and four concrete cutoff walls were constructed across the fissure, the farthest upstream located on the line of the main grout curtain. Two vertical grout holes were provided in each wall. The fissure was backfilled with impervious soil to the tops of the walls. All holes but one in the upstream wall were stage grouted starting with 25 lb. per sq. in. 5 ft. below the bottom of the walls and increasing by 5 lb. for each additional 10 ft. of hole. The remaining hole in the upstream wall was extended by means of a pipe through the embankment for grouting after completion of the fill.

Embankment.—The main dam embankment contains slightly over 5,200,000 cu. yards of earth fill. The dam prism, shown in Fig. 3, has a top width of 30 ft. at elevation 272 and slopes in an upstream direction at 1 in 3 to elevation 226, thence at 1 in 4 to the base of the dam. The downstream slope is 1 in $2\frac{1}{2}$ with 15 ft. wide berms at elevations 222 and 172. The top of the dam is cambered vertically a maximum of 3 ft. and horizontally 6 ft. in the upstream direction.

Prior to construction of the dam, soils investigations were carried on by the Irrigation Department. Most of the soil available for the embankment was a product of the weathering of the parent granite gneiss. Some pervious alluvium was present in the river bed. On the basis of preliminary field investigation the dam was designed as a zoned embankment, the cross-section consisting of a core of rolled impervious earth in the central third with zones of rolled semipervious material upstream and downstream of the core. A 3 ft. thick blanket of pervious sand drained by a concrete tile system was provided under the downstream semipervious zone. More extensive investigations revealed that any semipervious material which might be present at the site occurred in small pockets unworkable economically with modern heavy construction equipment. For practical considerations, the borrow pit material was all impervious, the only source of pervious material being clean coarse sand in the river bed. The latter material could be obtained by dredging with a clam-shell at high cost compared to the cost of impervious borrow. In order to utilise this material most effectively and efficiently, the sand blanket in the downstream portion of the embankment was extended upward into the embankment along the line of the downstream side of the original impervious core. At the same time the thickness of the pervious drain blanket was increased from 3 ft. to 4 ft. to increase its capacity. The inclined sand zone is intended to keep the line of saturation from emerging at the downstream face of the dam. The sand blanket alone would suffice for this purpose except for the probability of stratification in the fill. Even under favourable circumstances stratification develops in a rolled embankment, and in this case its probable extent was increased by the variability of the available borrow material. In effect an anisotropic condition will result wherein the permeability of the fill in a horizontal direction is many times greater (probably more than 10 times greater) than the permeability in a vertical direction. Seepage analyses by flow-net construction indicated that such stratification would result in the seepage line emerging on the face

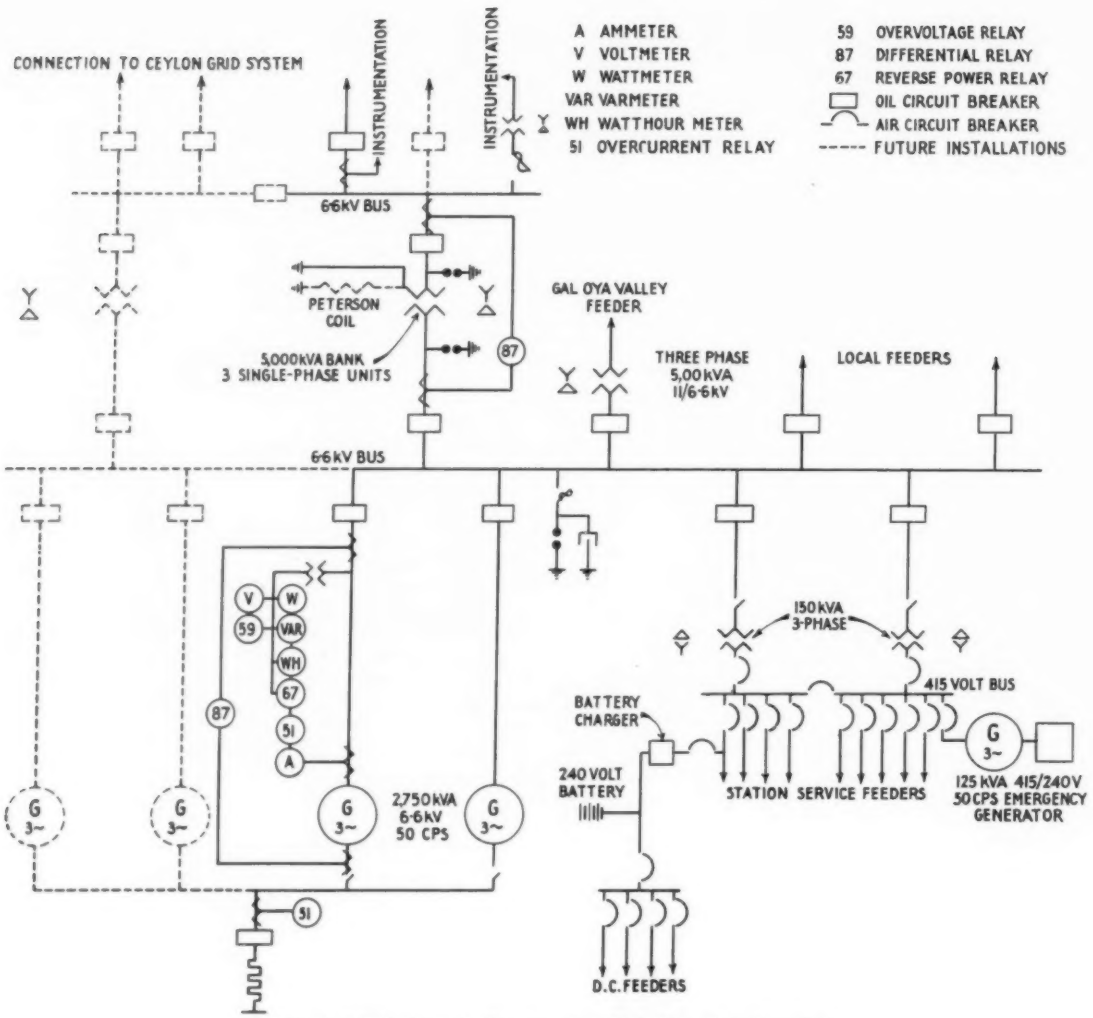


Fig. 4. Main circuit diagram for Gal Oya development

of the dam if the inclined sand zone were not provided. Stability of the embankment section was analysed by the Swedish circular-arc method using the results of soil tests performed in the soils laboratories of the Ceylonese Irrigation Department and the U.S. Bureau of Reclamation. A piezometer installation for measuring pore-water pressures caused by seepage through the dam and consolidation of the embankment materials, and a telescoping pipe installation for measuring the magnitude of this consolidation, were installed by the Soils Laboratory of the Irrigation Department. These installations are similar to those developed by the U.S. Bureau of Reclamation for their earth dams.

A 3 ft. thick layer of riprap underlain by a 9 in. gravel filter protects the upstream slope of the dam against the action of waves. The downstream slope is covered by 6 in. of sod obtained from existing tanks located nearby. An elaborate surface drainage system is necessitated by frequent heavy rains which occur at the site. Concrete drop inlets on the crest are drained by 12 in. concrete pipes carried down the slopes. Longitudinal gutters on the berms with cross-drains down the slopes carry the surface runoff to a collector channel at the toe of the dam which in turn discharges the surface runoff into the power-house tailrace channel. Gutters, cross-drains and the collector channel are lined with precast concrete sections. Sod was used rather than more effective rock riprap because of the desire of the Irrigation Department to obtain a more pleasing appearance. In further recognition of aesthetics, the area at the toe of the dam was built up to elevation 155 and attractively landscaped. A concrete parapet wall will be constructed on top of the dam several years hence when consolidation of the dam is nearly completed. This work will be performed by Irrigation Department forces.

Excavation in the borrow pits was performed by power shovels, belt-conveyor loaders and scrapers. In the case of shovels and loaders, the material was hauled to the embankment in 13 yard Euclid bottom-dump trucks. The maximum hauling distance was about two miles. Material was spread in 8 in. thick layers and compacted by double-drum sheepsfoot rollers. About 10 to 15 thousand yards of embank-

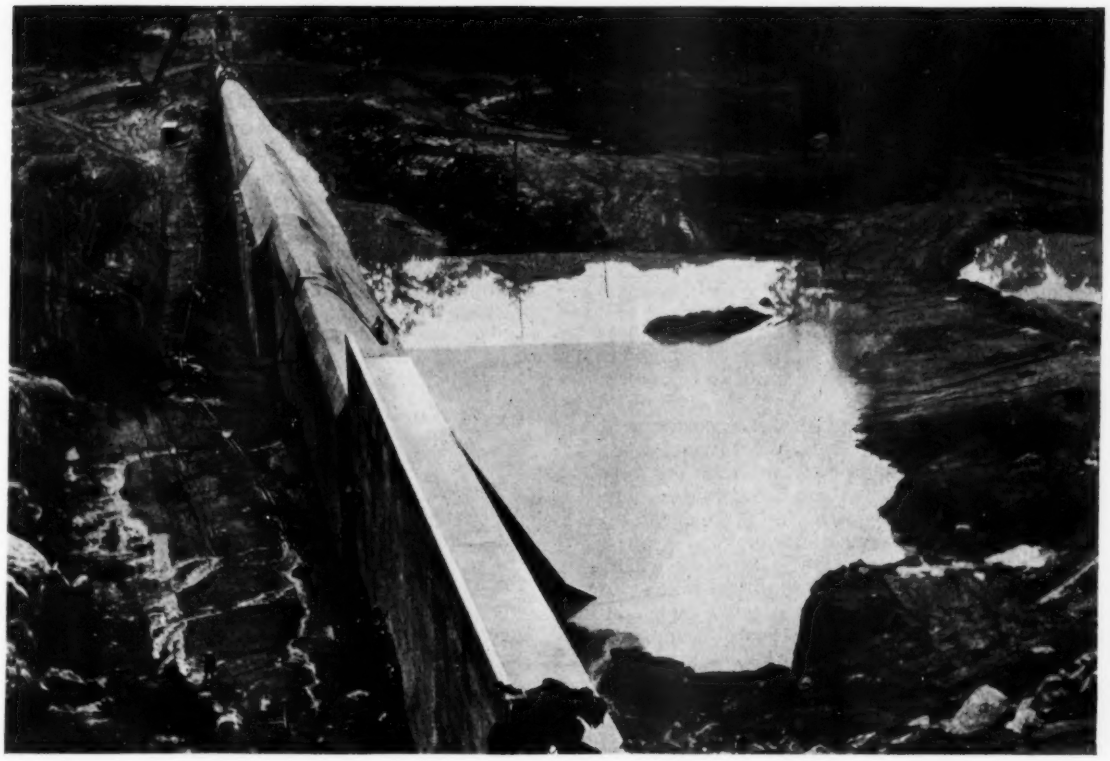


Fig. 5. View of completed spillway dam

ment were placed daily at the height of the construction season, operating on two ten-hour shifts. No material was placed in the dam at the peak of the north-east monsoons. Embankment placing operations commenced on June 1, 1949, and were completed on November 12, 1951.

Spillway Dam

Of the 1,050 ft. length of the spillway dam, 770 ft. is an uncontrolled overfall section (Fig. 5), with the remaining 280 ft. divided between two non-overfall gravity sections. The crest of the overfall section is generally at elevation 260·25. However, a 120 ft. length of the crest is at elevation 260·0 to localise small spillway discharges where they can most readily get away. The downstream face of the dam is sloped at 1 in 0·65 and the upstream face is vertical with a small batter near the base. The crest block of the non-overfall section is 10 ft. wide with the top at elevation 269 and a 12 in. thick concrete parapet extending upward 3 ft. to the top elevation of the dam. Transverse contraction joints divide the dam into 50 ft. wide blocks with the exception of three blocks near the centre, which are 40 ft. wide. Monel-metal sealing strips prevent leakage through the joints. Six 5·75 × 5·75 sluices through the dam with invert at elevation 212 provide for a maximum discharge of 7,000 cusecs with the reservoir at elevation 248. The sluices are controlled by hydraulically operated high-pressure slide gates. Power for operating the gates is supplied by a transmission line from the power-house, with an emergency power supply from a 15 kVA petrol-engine-driven generator.

The spillway is capable of handling a flood with a peak flow of 193,000 cusecs starting with a full reservoir. This would result in a peak discharge of 103,000 cusecs and a maximum pool level of 270·4. Only a simple bucket is provided at the toe of the overfall section and no special provisions are made for energy dissipation. Since the toe of the spillway dam will be open for inspection and possible repairs throughout the dry season, measures for energy dissipation will be taken only if they are shown to be necessary after initial operation of the reservoir.

The spillway dam was constructed entirely in the dry. To obtain a satisfactory foundation it was necessary to remove as much as 60 ft. of overburden. The foundation was grouted using a line of grout holes 30 ft. deep on 10 ft. centres. Excavation for the foundation started in November, 1949. The first bucket of concrete was placed on February 3, 1950, and the final bucket on January 5, 1951. Specifications called for a compressive strength after 28 days of 2,500 lb. per sq. in. for the non-overflow sections and 3,000 lb. per sq. in. for overflow sections. A total of 44,483 cu. yards of mass concrete was required for the spillway dam and 108 cu. yards of additional concrete for training walls and parapets. About 6 to 7 thousand cu. yards per month were placed in the dam at the peak of operations. The specifications permitted a maximum concrete temperature of 85°F. when depositing in the forms. As a consequence of high daytime temperatures prevailing at the site, concrete was poured only at night in a single 12 hour shift. The completed spillway dam is shown in Fig. 5.

(To be continued)

Book Reviews

A Treatise on Applied Hydraulics. Fourth Edition. By Herbert Addison, O.B.E., M.Sc., M.I.C.E., M.I.Mech.E. Published by Chapman & Hall Limited, 37, Essex Street, London, W.C.2. 1954. 8¾ in. x 5½ in. x 2 in. 724 pp. 475 ff. Price 56s. net.

This book, when it was first published 20 years ago, immediately made a name for itself by the clear way in which it dealt with the fundamentals of hydraulic theory and practice, and still remains one of the most valuable books in this field, at any rate in the English language. The fourth edition has been revised and considerably enlarged to encompass the progress that has taken place in the intervening years, but no change has been made in the style of presentation, which is distinguished for its lucidity and logical sequence. All hydraulic principles are explained with admirable directness, and their applications in practice briefly described, while an unusually complete bibliography provides an excellent guide to more detailed sources of information.

The work is divided into two parts, which deal respectively with fundamental principles and practical application. The first part considers liquids and their properties, liquids at rest, liquids in motion, flow through orifices and over weirs, flow through closed conduits, flow along open channels, dynamic pressure of liquids, and radial and rotary motion of liquids. The second part deals with pipes and pipe systems, control of water in open channels, the construction and performance of hydraulic turbines, pumping machinery of all kinds, hydraulic transmission and storage of energy, and hydraulic measurements. More than 100 pages are devoted to example questions and answers which should prove most valuable to students. A further feature is a brief biographical note on some of the mathematicians, scientists and engineers who have contributed so much to the knowledge we now possess of hydraulic engineering. There is also an excellent index.

Vitesse Critique des Arbres en Rotation (Critical Speeds of Revolving Shafts). By Lucien Borel, Ingénieur E.P.U.L. Published and printed by Imprimerie La Concorde, Terreaux 29-31, Lausanne. 6·2 in. by 8·2 in., 152 pp., 50 ff. Price in Switzerland, paper-wrappered, Sw. Fr. 9·85.

The study of the critical speeds of revolving shafts is generally based on the assumption that rotary motion is brought about according to a constant rate of speed; it is, however, obvious that, besides starting and stopping, the rotary motion of a shaft is influenced by various accidental perturbations, so that the hypothesis of a constant rate of speed systematically eliminates any consideration of the dynamic stability of motion. If the behaviour of a revolving shaft is to be thoroughly investigated, it is therefore advisable not to neglect the importance of transient conditions which cannot possibly be known beforehand. The whole problem is exhaustively dealt with in this neat little brochure which presents a synthesis of the investigations of numerous research workers on the subject. The work is divided into two sections:—Theoretical research, and Methods of practical calculation. An intuitive exposition gives the theoretical aspects of the problem; the physical nature of critical speed is explained in the light of the general theorems of mechanics. In the practical section, all the methods at present in use in engineering are analysed and thoroughly discussed, separate sub-sections being devoted to two-bearing and three-bearing shafts. This study will be welcomed by the designer, who has hitherto been confronted by an abundance of methods which, too frequently, were presented in fragmentary form or dealt with the details of problems of too special a nature. A short bibliography and an analytical index complete the volume.

Design Criteria for Concrete Gravity and Arch Dams. By F. D. Kirn. Engineering Monograph No. 19 of the United States Department of the Interior, Bureau of Reclamation, Building 53, Denver Federal Center, Denver 2, Colorado, U.S.A.

The Bureau of Reclamation issues limited editions of monographs from time to time with the object of recording developments, innovations and progress in the various branches of engineering which fall within the limits of its activities. The present monograph presents the basic data which were formulated by a panel of eleven engineers, whose object was to make special studies to ensure that the Bureau's procedure in connection with concrete gravity and arch dams should be consistent with the best existing knowledge and concepts. It is considered that adequate data are not yet available for a wholly rational approach to all phases of design, and that no criterion may be properly applied under all conditions; in consequence each specific criterion given in the monograph is preceded by an explanation of its limitations when applied to special conditions. The subject matter is discussed under Basic design considerations; Factors relating to stability (dead load, shear resistance, quality and strength of concrete); Configuration of dam and foundations; Structural competency of foundations and abutments, etc.; and Instability (reservoir and tailwater loads, uplift, earthquakes, and ice). Further sections deal with Grouting contraction joints in straight gravity dams, Load combinations, Factors of safety and the Overturning of gravity dams.

Previous monographs have been prepared on the following subjects, and are available to interested engineers:— (1) Petrography and engineering properties of igneous rock; (2) Multiple correlation in forecasting seasonal run-off; (3) Welded steel penstocks; (4) Project effect on South Platte River pollution; (5) Soap Lake siphon; (6) Stress analysis of concrete pipe; (7) Friction factors for large conduits flowing full; (8) Theory and problems of water percolation; (9) Discharge coefficients for irregular overfall spillways; (10) Reinforced concrete design data; (11) Hydrodynamic pressures on dams due to horizontal earthquake effects; (12) Stresses around a gallery determined by the photoelastic interferometer; (13) Estimating foundation settlement by one-dimensional consolidation tests; (14) Beggs deformeter stress analysis of single-barrel conduits; (15) Photoelastic stress analysis of bridge piers; (17) Surge control on the Coachella pipe distribution system; (18) Hydraulic laboratory practice.

Correspondence

Forçacava* Hydro-Electric Scheme

To the Editor of WATER POWER

There appeared in the Sept., Oct. and Nov. issues, 1953, articles by Prof. Dr. Techn. L. V. Rabcewicz describing in detail the methods of rock excavation for the "Forçacava Hydro-Electric Scheme" in Brazil which consisted of a large underground power cavern, pressure shaft, tailrace tunnel and related structures. Prof. Rabcewicz should have pointed out that neither he nor his company, Sentab, were responsible for all the methods adopted, but they were worked out during the progress of the work in collaboration with the engineering staff of the Rio de Janeiro Light and Power Co. Ltd., the owner of the project.

The writer was engaged by the power company for more than five years as an engineering geologist and is familiar with all aspects of the development, exploration and excavation by Sentab. The power company's personnel were not inexperienced in this area or type of rock encountered since they had previously constructed more than ten kilometres of large-diameter tunnel under similar conditions.

The site for the underground power cavern and related features was explored by several thousand feet of diamond core drill holes penetrating to the full depth of the proposed structures during a period of two years prior to construction. The fundamentals of this geologic investigation have been adopted by Prof. Rabcewicz in his Fig. 3, section A—A, without acknowledgement as to its source.

Prof. Rabcewicz is incorrect in his statement when he states that "the gneiss was often intersected by fault zones of various thickness, mostly more or less parallel to the stratifications." Gneisses do not have stratifications since this term is limited to sedimentary rocks and a gneiss is a metamorphic rock. He has reference to a type of rock alteration known to geologists as hydrothermal or kaolinisation. A minor amount of faulting has taken place, but faulting or crushing is non-existent in the majority of the kaolinised zones. Hot ascending waters decomposed the gneiss in place. The larger zone of kaolinised gneiss in the power cavern had developed along a fault one to two metres wide but the hot waters later decomposed the rock on either side for approximately 5 metres. Slight movement in this zone after the kaolinisation developed several prominent slickensides near the base of the zone as shown by section A—A, Fig. 13, in Prof. Rabcewicz's article.

The supporting value of columns 1 to 8 in the south wall as shown by his Fig. 13 is of doubtful value because they are poorly anchored at both top and bottom. The material in this area would have required no supports had it not been for the slickensides near the base of the zone. It remained standing without any supports for many weeks and columns 1 to 3 were first constructed too small and were blasted out and replaced by larger columns. Since the slickensides were restricted to the bottom of the zone, it was not necessary to construct columns 20 metres high to support a zone only a few metres thick. It could have been accomplished faster and more economically by mining out the soft material along the slickensides in steps as the excavation progressed downward and replacing by concrete. Also, high-pressure grouting of the zone in advance of excavation would have greatly improved the character of the material, but was rejected by Prof. Rabcewicz because he did not think, at the time, that the material would cause any difficulty in the excavation of the cavern. High-pressure grout could have been forced into the shear planes and thereby greatly increasing the shearing resistance along them. There was ample time to accomplish this while the roof arch was being constructed. This could have been done by drilling angle holes under the abutments for the arch before the rock in the main part of the cavern was excavated.

Some supports, such as the horizontal strut shown in Fig. 15, were placed after the major portion of the excavation had been completed to become a part of the main permanent floor, at the owner's request as additional insurance against possible movement of the south wall between columns 1 to 8. This was believed to be desirable because little faith could be placed in the anchorage of these columns.

Prof. Rabcewicz states under the heading of "North Wall" that "the plastic kaolinised zone K suffered a deformation caused by its own weight and the weight of the superimposed layer f_1." This material was not plastic since it contained no clay. Zone K is shown in his Fig. 18 and if it were plastic, it would not stand as shown by this figure. After the cracks appeared in the concrete arches and overlying rock, a field examination clearly showed that it was a result of inadequate support of the arches and rock. The arches had been placed on soft material as shown in his Fig. 16, section B—B and much of the soft material had been removed by later excavation as shown in his Figs. 18 and 21. The contractor was requested to place support V, Fig. 16, section B—B, immediately after the examination indicated the cause of the cracking, although he did not think it was necessary. A serious cave-in would have taken place here in a few days if this support had not been placed when it was.

While the rock in the draught tube area of 2 and 3 is not of the best type, it is not as bad as indicated by Prof. Rabcewicz. I recall that a large blast set off in draught tube 3, after the cracks appeared in the concrete of the arch, completely removed all supports and the area remained unsupported for more than two weeks with relatively little material falling into the excavation. A large part of the difficulty with the rock in this area resulted from the attempt to construct the arches from the top down rather than from the bottom up. This method did not keep the ground properly or tightly supported, which is a necessity. The material did not develop heavy loads on the supports and there was no reason why it could not have been done in the conventional manner.

Prof. Rabcewicz states under the heading of "Draught Tubes" that "it was first intended to proceed gradually with the excavation and concreting at three levels." It is strange that such an idea was never discussed with the engineering staff of the power company nor were the designs made with this in mind. It could not have been accomplished for a number of reasons. In the first place, the contractor was not equipped to place concrete under such conditions if he had been permitted to by the owners. The concrete could not have been placed from the top downward

* This power station has now been officially renamed "Nilo Peçanha," as announced on page 274 of the July 1954 issue of WATER POWER.

with satisfactory results for physical reasons and it would have been seriously marred by later blasting. The thin walls were not designed to take loads and they could have been seriously damaged by later blasting.

PORTLAND P. FOX

Member, American Soc. Civil Engineering
Fellow, Geological Society of America.

To the Editor of WATER POWER

Regarding Mr. Portland P. Fox's letter I wish to make the following statements.

Mr. Fox is right regarding the use of the word "stratification"; the term "cleavage" should have been used instead.

The purpose of the columns to support the roof, to stop the rock from sliding down and to prevent the progressive loosening of the bad zone along the slickensides, was fully accomplished, and their anchoring was in correspondence with their bending strength. They could not be founded on a rock, the joints of which were dipping 45 to 55 degrees towards the power cavern but had to be brought down to the bottom.

High-pressure grouting of the bad zone before starting the excavation may perhaps have helped to some extent to improve the situation at the south wall but could not have influenced fundamentally the loosening effect by detonation waves.

Mining out the soft parts and replacing them by concrete would not have been possible because the concrete resting on inclined planes would move down when blasting out the deeper portions. This is what happened to the columns 2 and 3 which were originally designed shorter and had to be replaced. The proposal would be feasible if the whole wall had been concreted, bridging horizontally over the bad portion and deeply anchored. But in this case the proposal would be identical with Sentab's suggestion mentioned at the end of this rejoinder.

The zone "K" in Fig. 16 was absolutely plastic, containing a large quantity of clay in its numerous joints. This can easily be judged from the fact that shortly after excavation a crack always opened along this zone and the soft, i.e. "plastic" material moved towards the open space, the overhanging pieces falling off gradually, disclosing the upper slickensided joint as seen in Fig. 20. Plasticity is a relative conception always referable to the forces at work.

The writer did not oppose the setting of strut *V*, which certainly fulfilled its purpose but was of secondary importance as the heavy strut IV took the main load of the layer *f*. A cave-in in draft tube 2 would never have occurred as the abutments of the arch were heavily reinforced and a good deal of anchoring placed. There was no imminent danger as the movements were stopped immediately after the placing of strut IV. The arches were placed on soft material with the intention of supporting their abutments, as soon as feasible, by columns from the bottom. This was certainly the right way to do the job under the conditions prevailing. Starting the draft tubes after the excavation had reached the bottom would naturally have necessitated the same support by struts to keep the layers *f* in place, but would have caused a large overbreak as the whole layer *K* would have come down when excavating the draft tubes. A way out would have been to excavate a small shaft along the surface of the wall and to brace it, but this would have been very expensive and much slower.

The method of proceeding gradually downwards in three steps, excavating and concreting, was proposed by Sentab and in fact discussed in detail with the project manager and his staff in August 1951. It is strange that Mr. Fox, who was not present at these conferences, was not informed. The method could of course have been successfully used, the damaging of concrete being prevented by covering the blasting area by steel cable mattresses and by blasting carefully. The concreting equipment available was efficient enough to do the job intended.

The article was written with kind permission of the management of the Brazilian Traction, Light and Power Co. Ltd. and the writer thought that the acknowledgment thereof would automatically cover the use of the drawings Figs. 3 and 4. It is not usual in European technical journals to mention the names of the leading staff. The article certainly has not the intention of questioning the qualities of the owners' staff, and as regards Mr. Fox's part, the writer stated the geological investigations to have been carried out in an exemplary manner. I wish to point out that the collaboration between the owners' and Sentab's staff was extremely intimate. The writer regrets the misunderstandings leading to Mr. Fox's letter.

PROF. DR. TECHN. L. V. RABCEWICZ.

"Micro" Water-Power Plants

To the Editor of WATER POWER

I have read with great interest the Editorial in your July issue under the above heading.

Already considerable numbers of small hydro-electric sets of the type which you mention have been installed in this country, and the North of Scotland Hydro-Electric Board have, as you indicate, been most progressive in this development.

The majority of water supply storage reservoirs are required by law to pass a certain amount of compensation water continuously and nearly all the authorities responsible for these reservoirs have installed hydro-electric plant for local supply. An interesting example is the City of Birmingham Water Department who have recently commissioned two 162·5 kW hydro-electric sets, using the compensation water from the Elan Valley Dam. These units normally operate under hand control, feeding as much current as possible into the network of the South Wales Electricity Board, but are provided with governors enabling them to supply the local system independent of the grid should this be necessary.

Other potential sources of small water power development are on sites once occupied by the water wheels which provided power for factories or corn mills. Some of these are derelict, but the weirs, races and wheel pits are in many cases in good condition, and turbines driving induction generators could economically be installed.

As you rightly point out low capital cost and extreme simplicity are of primary importance, but I do not think that fixed guide turbines would be satisfactory in this country due to the very wide variation in the flow of our streams and rivers. It would almost certainly be necessary to regulate the guide vane setting automatically from the head water level so as to ensure that the maximum head was always maintained.

It is unfortunate that throughout the English countryside many fine old corn mills are at present falling into decay, and at the same time water power capable of generating millions of units per annum is being wasted. It is to be hoped that at some time the British Electricity Authority will decide to investigate this field, bearing in mind the fact that, although such plant cannot be regarded as "firm power," our rivers have their greatest run-off during the winter months. If, in addition, some of the old water-mills could be preserved we should all have cause to thank them.

PAUL N. WILSON.

Canal Ironworks,
Kendal, Westmorland.

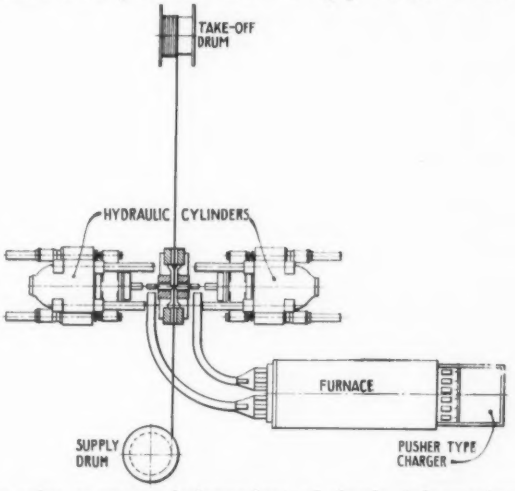
Extruded Aluminium Cable Sheathing

A significant advance in the technique of cable manufacture has been made at the Prescot works of British Insulated Callender's Cables Limited by means of which it is possible for a power cable to be covered with an extruded sheathing of aluminium for any desired length.

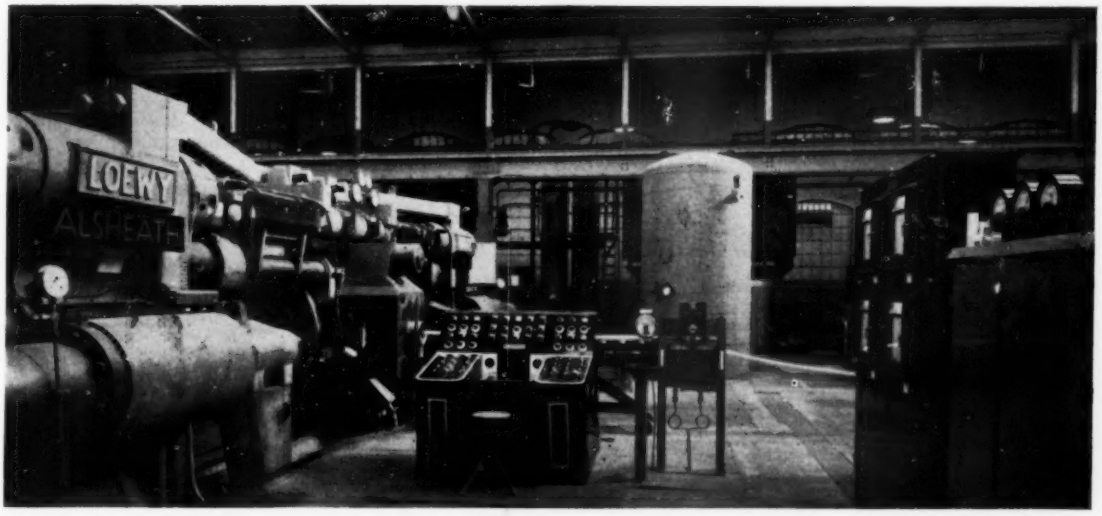
Direct extrusion of aluminium sheaths has long been a desired end to cable makers and after close study of the problems involved, BICC concluded some years ago that the following conditions must be satisfied if the extrusion of aluminium sheaths could take its place as a normal production process: (a) The use of commercial grades of aluminium; (b) The use of solid billets; (c) Low extrusion temperatures, preferably not exceeding 300°C.; (d) The ability to produce sheathed cables in long continuous lengths; (e) Satisfactory welds between successive billets; (f) Minimum charging time to prevent overheating of the cable while stationary in the press; (g) Avoidance of deformation and metallurgical features at the point on the sheath which is within the die when the cable is stationary for recharging.

No existing type of press was capable of meeting all these desiderates and BICC and the Loewy Engineering Co. Ltd. co-operated to design one. The resulting press is believed to be unique. It is hydraulically operated with two horizontally opposed cylinders each 45 in. in diameter, operating at 4,280 lb. per sq. in. pressure, and each giving norminally 3,000 tons capacity. Twin billets are extruded through a common core and die to give sheaths ranging from 1·4 in. diameter to 3·5 in. dia.

The cable passes through the press at right angles to the axis of the cylinders, this permitting the use of solid billets and reducing to a minimum the length of cable exposed to the heat of the press. The extrusion tools are mounted in such a way that they retain their positions and the gap between them



remains constant, independent of elastic deformation due to expansion or contraction under heat. The extruded sheathings are therefore free from any weakness at "stop-marks." Special controls ensure that the speed of extrusion from the two containers is constantly balanced. Mechanical loading facilities have brought down the recharging time to a matter of seconds. Special attention has been devoted to the prevention of air inclusions. All major movements



are controlled by servo-operated valves from a central pulpit providing for fully automatic control or for individual control as desired.

Trials to date indicate that all the initial objectives are being met and the press is approaching the stage when it will take its place as a routine tool in cable manufacture. Enough cable has been sheathed to show that the "stop-marks" are satisfactory from the dimensional and metallurgical aspects; they comply with the appropriate bend test requirements and they can withstand high hoop stress. The welds formed between successive billets (as distinct from "stop-marks") are not discernible in the extruded sheath.

With aluminium of 99·8 per cent. purity and extrusion temperatures of 280° C., the extruded sheaths are in the lightly annealed condition. Tensile strength is 5·0 to 6·0 tons per sq. in., and elongation on a length of two inches in the longitudinal direction of the sheath is 30 to 50 per cent., depending on diameter and thickness. The grain size is fine at all parts of the sheath, including the "stop-marks." These properties ensure that the springiness associated with cold working is largely eliminated.

Much remains to be done in equipping the press with the tools required for a comprehensive range of cable size and in proving its performance over that range, but sufficient has been done to indicate that we have here a practical tool which meets a basic need.

Sandvik Flexible Drill Steels

During a recent visit to the Stockholm works of A.B. Atlas Diesel we were given a demonstration of the use of flexible steels to drill long stoping holes in confined spaces. The hole is commenced in the usual way with a normal hexagonal steel, but instead of extending the steel by jointing successive sections it is replaced by flexible steels of progressively increasing lengths. The steels are laid on the floor—preferably on a light channel iron so that they can slide easily without obstruction—and are of a rectangular section such that they can be flexed sufficiently to enable the steel being used to be fed into the hole. Changing steels is a very simple matter and the time required for making and breaking joints, as would be needed for hexagonal steels, is eliminated.

These steels are made by the Sandvik Steelworks Company of Sweden, and are available in lengths up to 6 m. (19 ft. 8 in.). They have central flushing holes and tungsten-carbide inserts similar to the standard Coromant drill steels made by this firm.

The accompanying illustration shows a Sandvik Coromant flexible drill steel in use in Grangesberg iron mine in central Sweden. Blast holes 6 m. long are being drilled by this method, and we understand that both management and drillers are well satisfied with performance.

Mobile Weld-testing Equipment

Pilgrim Mobile Units Ltd. have devised a mobile radiographic and weld-testing laboratory which incorporates spacious dark-room facilities, an office and a testing section, and is now in service with Welding Supervision Limited. To withstand the rigorous conditions encountered on remote sites, the chassis has been specially strengthened, and the layout arranged to achieve even distribution of load at all times.

The dark-room is adequately ventilated to permit the welding engineer-radiographers to work under all climatic conditions. A sliding door to the dark-room is fitted with a warning light and automatic lock, as a precaution against interruptions during processing. Two water-storage tanks provide a supply of water for the sink, while electrical heating elements maintain the correct temperature for developing chemicals. Acid-resistant plastic splash panels have been used for the wet and dry benches, and the floor, which has good drainage, is surfaced with thick rubber sheeting as a protection against corrosion from spilled chemicals.

The forward compartment of the vehicle is fitted with adequate storage facilities and a work bench so that welders' test specimens can be ground and filed to the correct dimensions. A foldaway drawing board is available for technical drawings and sketches, and the X-ray unit can also be stored here. Mounted over the reinforced wheel box is a portable tensile, testing and bending machine capable of pulling test specimens up to 15 tons direct load. The unit carries radioactive isotopes and portable X-ray sets as well as ultrasonic testing equipment. The isotopes, in their containers, are stored in a compartment lined with ½ in. lead sheet which provides additional screening to the operator and permits the safe handling of undeveloped films in the workshop and office sections.

River Crossing by Rocket Apparatus

Where the Uganda Electricity Board's new 132 kV overhead transmission line, from Owen Falls Power Station to Tororo, crosses the Nile, the river flows swiftly over a rocky bed. The resulting rapids make the crossing of the river a hazardous undertaking, and even if the crew of a canoe or small river craft escaped

death from drowning in the rapids if they were overturned, they would be in dire danger from crocodiles which abound in the still waters downstream.

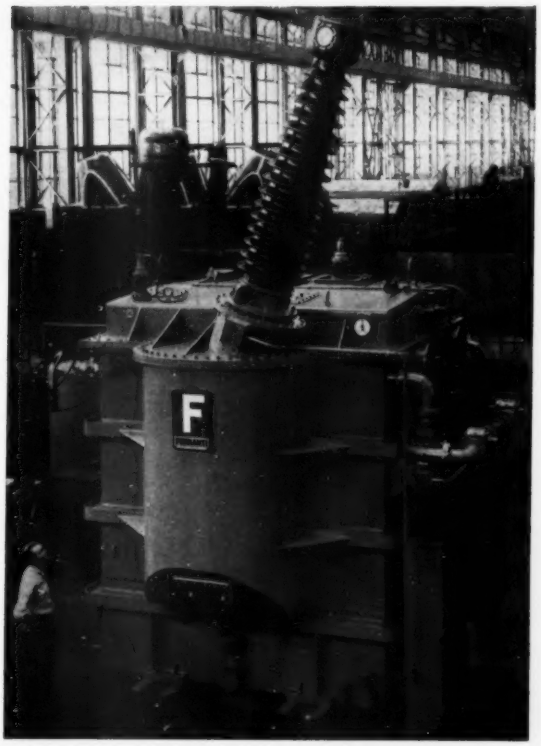
To establish a line of communication between the two banks of the river, the overhead-line contractors, British Insulated Callender's Construction Company Limited decided to make use of a Schermuly rocket apparatus which was obtained on loan from the port authorities at Mombasa. In the first instance a light line, forming part of the Schermuly equipment, was fired across the river. This line was then used to draw over a half-inch diameter manila/sisal compound rope which was made-off to a convenient tree on either side of the river with sufficient tension to keep it clear of the water. This rope was subsequently replaced by a stranded steel draw wire which will later be used to pull the steel-cored aluminium conductors across the river. At the point of crossing the river is 1,500 ft. wide but the rocket was fired at an angle of approximately 30° to the final route to land on a promontory on the river bank—a distance of 800 ft. The point from which the rocket was fired is 120 ft. above river level; on the far side, although the general mass of the ground rises sharply from the river, the promontory to which the rocket was fired is not much above river level. The contractors have previously made use of standard coastguard rocket-firing apparatus to cross rivers and gorges, but this is the first occasion on which they have used the Schermuly apparatus.

The unit can also carry magnetic and fluoroscopic apparatus if required.

Transformers for the Garrison Dam

Ferranti Ltd. are supplying a total of nine 33,333 kVA, single-phase, 60 cycles, 13,200/230,000 volt generator transformers forming three 100,000 kVA, three-phase banks to the U.S. Army Corps of Engineers for installation in the Garrison Dam power plant in North Dakota, U.S.A.

The power plant is located at the west end of the Garrison Dam on the Missouri River about 70 miles upstream from Bismarck, North Dakota. The generating plant includes five 80,000 kW hydro-electric units with a maximum continuous rating of 97,800 kVA each. Generation is at 13,800 V which will be stepped up for transmission to 115,000 V and 230,000 V by transformers located outdoors at the power plant on a transformer deck built on the downstream side of the power house and over the draught tubes. Three generators will connect to the 230,000 volt bus, and two to the 115,000 volt bus. The 230,000 V Ferranti transformers are of the oil immersed, inert-gas FOW type fitted with off-load tapping switches. They comply with the NEMA and ASA standards, and



noise and impulse level tests have been carried out to the requirements of these standards. The impulse tests corresponded to a full wave test of 900 kV and included front-of-wave tests with a crest voltage of 1,240 kV. The total weight of each transformer bank is 216 tons and the weight of each transformer when stripped for transport is 51·5 tons. It is understood that the total value of the contract is $880,000.

Abstracts from the World Technical Press

Transmission Line Construction Problem

This article describes an investigation of the difficulties experienced in tensioning the transmission-line conductors over steep ground in New Zealand. The transmission line in question runs from Cobb power station, at the junction of the Cobb and Takaka rivers, to Nelson, and from Nelson to Blenheim. After leaving the station, the line rises 1,210 ft. to a narrow ridge, and then by a 65-chain span crosses a gorge to a relatively flat plateau about 2,200 ft. above sea level. At five miles from the station, the line descends a steep ridge to Upper Takaka, 300 ft. above sea level, to rise again very soon to the top of Takaka Hill (2,600 ft.). A detailed analysis of the factors affecting the Takaka Hill section, whose tensioning proved particularly awkward, was carried out and the conclusions reached were subsequently applied to the Nelson-Blenheim line, where conditions are similar. Determining the correct clamping point of the conductor at each structure proved the most difficult problem. Fig. 1(a) shows that when the conductor is clamped in at structure 48, and span 48-47 is correctly tensioned, the insulators will move downhill but will fail to become vertical by about 8 in. After consider-

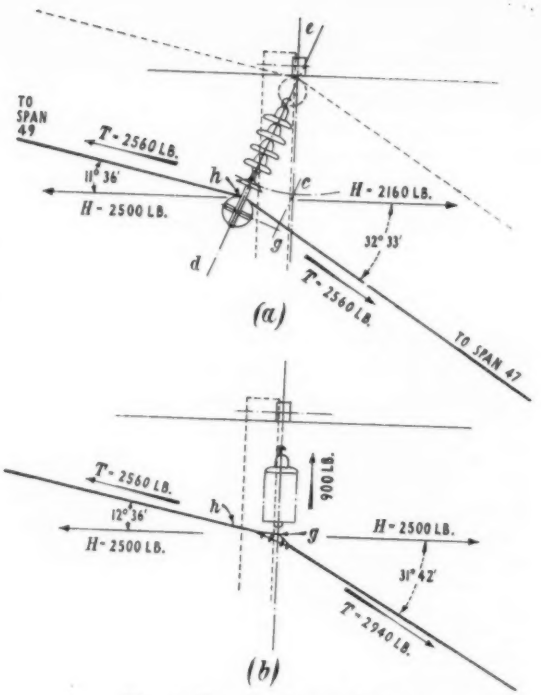


Fig. 1. Tension in conductor

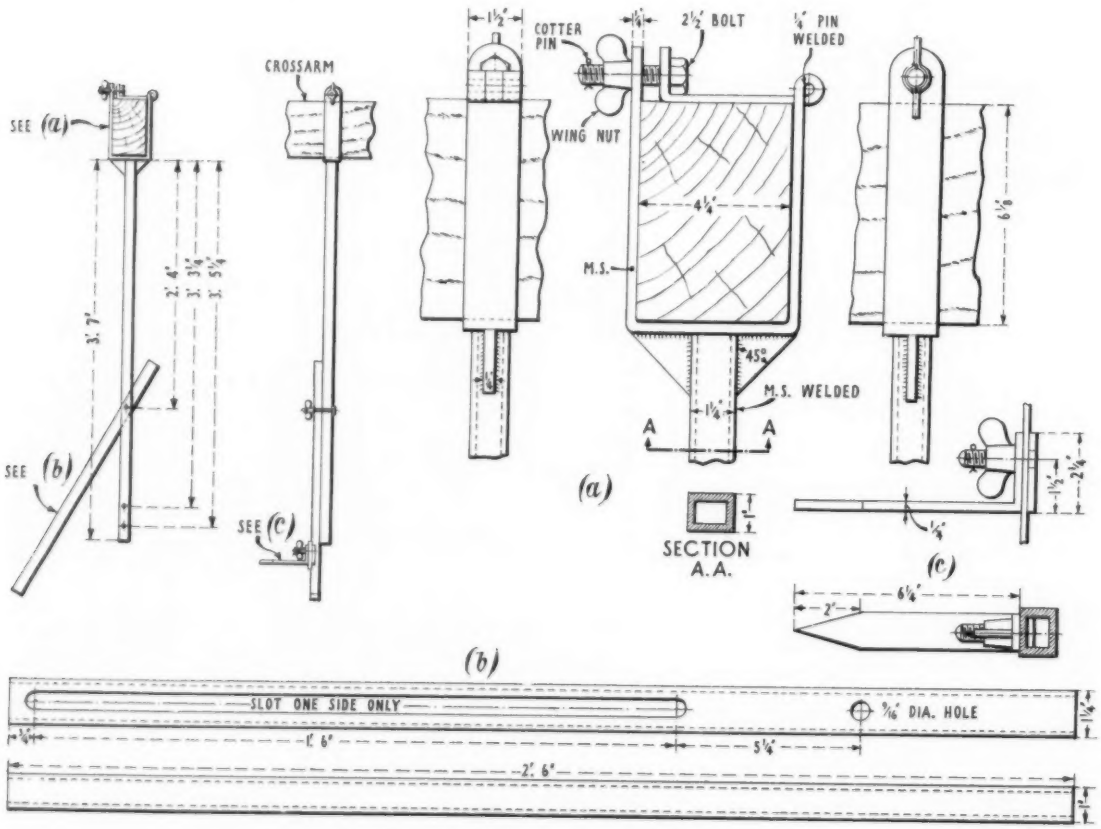


Fig. 2. Tool for indicating clamping point

able experience, the field staff were able to estimate the correct clamping point g so as to ensure the verticality of the insulators. The device shown in Fig. 2 enables point g to be easily determined by reproducing lines e—c and c—g. The movable arm with the pointer attached is located so that it pivots about a point which is the same distance from the cross-arm as the conductor will be when clamped. After the conductor has been tensioned, but before it is clamped, the movable arm of the device is placed parallel to the centre line of the inclined string of insulators, and the correct clamping point is shown by the pointer. A device of this kind was used when clamping a steep section of the transmission line from Stoke to Blenheim. When, beginning with the top span, all the spans had been tensioned, the tensioning of each span was checked and was found to be correct, no adjustment being required. (E. B. Mackenzie, *New Zealand Engineering*, Vol. 8, No. 10, October 15, 1953, p. 354, 4 pp., 9 ff.)

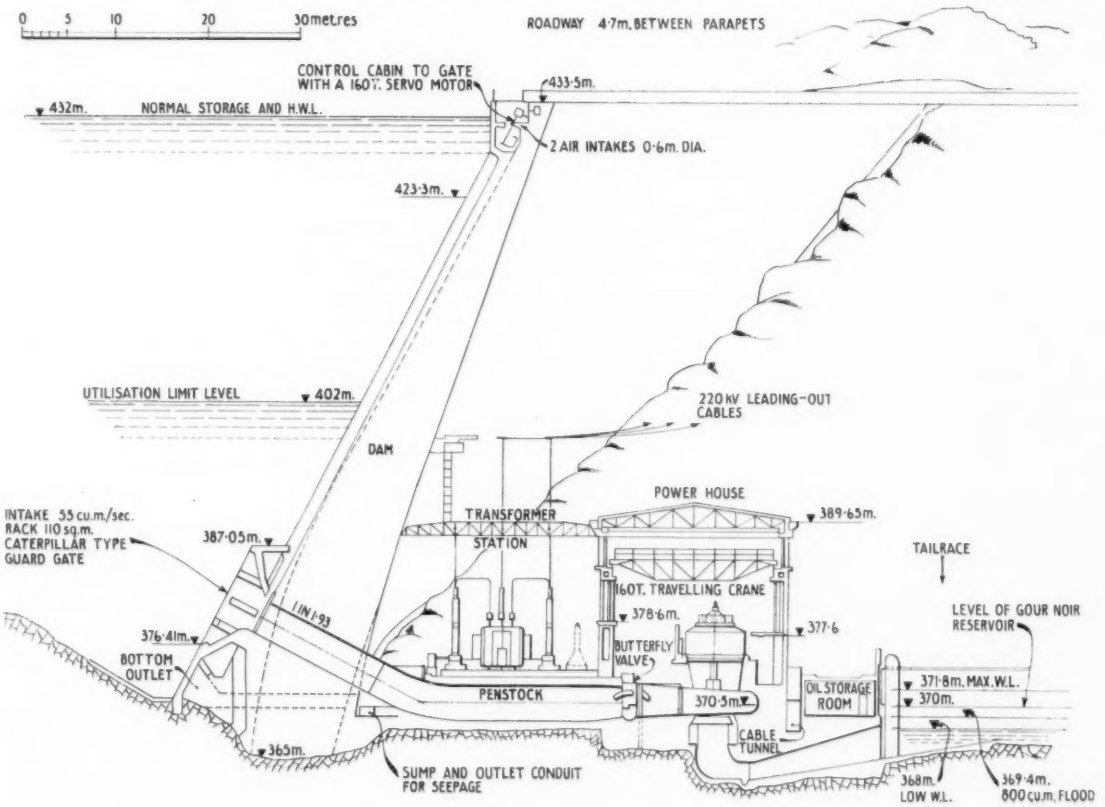
Maronne Development, France

The Maronne river is one of the major tributaries of the Dordogne, into which it flows not very far downstream of the Chastang Dam. The main work of this development is the Enchanet Dam, whose main purpose is to ensure an accumulation of 73 million cu. m. of water above the St-Geniez-ô-Merle power station, which is further downstream in the Maronne valley. The power house erected at the foot of the dam utilises the head created by the difference between the water level in the Enchanet reservoir (432 m OD) and the storage level at the Gour Noir dam, which constitutes the intake of the St-Geniez plant (370 m OD).

The Enchanet dam is a thin arch of the cantilever type; the batter of the upstream face is 1/2 at the central block. The total height of the structure is 68·5 m., its radius 104 m. at the crest, which has a length of 230 m. The thickness, 11 m. at the foot of the dam, is only 2 m. at the crest, which carries a 3 m. wide overhanging roadway. Owing to the uncommon design of this dam, the volume of concrete involved was relatively small. In arched dams built in horizontal slices, the endeavour to reduce the volume of concrete to a minimum leads to the plotting of each slice in such a way that the angle in the middle remains in the vicinity of 110 to 120°; when these slices are piled up one above the other, overhangs are unavoidable.

Were the central element of the dam vertical, the overhangs would be upstream at the springing of the arch, and stability, when the reservoir is empty, would be difficult to achieve. On the other hand, with the central element bending downstream, the overhangs are shifted to the downstream side, where they can, however, cause no major inconvenience seeing that the structure, owing to its being built in fairly horizontal slices, wedges in by its own weight and ensures stability.

The power house shelters a single vertical-axle generating set of 31,500 kVA, and the 11,000/220,000-V transformer is sited between the dam and the power house. The leading-out cables connect direct with the 220,000-V Gatelier-Le Breuil line, which passes near the works. Provision is made in



the switching gear for future operation by remote control from a neighbouring plant.

The underground spillway tunnel on the left bank, designed for a discharge of 800 cu. m. per second, has a slope of 52·5 per cent. and connects at the bottom with the temporary diversion tunnel. Work on this development started in the summer of 1946: concreting of the dam (65,000 cu. m.) was effected and completed in 1949. The plant is now in operation. (*La Houille Blanche*, Vol. 8, No. 5, October 1953, p. 656, 11 pp., 28 ff.)

Annular-Flow Turbines

The firm of Sfindex have recently published a reply to the article on the Bell annular-flow turbine (see abstract in WATER POWER, July 1954, p. 280) and maintain that their new turbine is essentially different in design. It is pointed out that there is nothing new in the idea of a full-admission impulse turbine, designs having evolved via the old Fourneyron and Girard types, and the more recent Reiffenstein design (Swiss Patent No. 115,379). However, in contrast to all its predecessors, it is claimed that the Sfindex guide-apparatus is the only one having a low-loss jet which can be regulated at all flows to give a constant whirl-velocity. In consequence, as the runner blades always

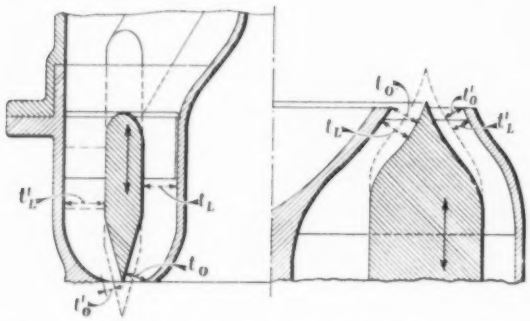


Diagram showing characteristics of Bell design (left) for securing annular flow and Sfindex design (right)

work under constant inlet conditions, " shock " losses are avoided and high efficiency at part load is maintained.

It is stated that the German patent (No. 452,983) of Hermann F. Bell specifically mentioned that the objective was to produce a full-admission impulse turbine using known types of guide-apparatus. The designs quoted by Bell do in fact show conventional guide-apparatus. The first example showed a Pelton nozzle with internal swirl vanes which would apparently cause the jet to disperse before hitting the runner. The second example illustrated had no swirl vanes, the flow-rotation being set up in a spiral casing similar to that in the Reiffenstein turbine except that it would not be expected to give constant whirl-velocity for various nozzle openings.

In an earlier Sfindex article (Technische Rundschau Bern, 17.7.53) Widmer showed that for a particular swirl vane setting and nozzle geometry, the whirl angle of the issuing jet is proportional to the ratio t_L/t_o. Corresponding parts of the Bell and Sfindex designs of guide-apparatus (see WATER POWER, July 1954, p. 280) are compared in Fig. 1 and although there is a superficial resemblance, they differ in one important respect. In the Bell design the ratio t_L/t_o varies with nozzle opening whereas in the Sfindex design, t_L/t_o is approximately constant so that the whirl angle does not vary with flow. This hydraulic advantage is gained by differences in nozzle geometry due to the Bell design having swirl vanes which slide with the ring-shaped needle, whereas in the Sfindex turbine the needle effectively slides round the fixed swirl vanes.

Having demonstrated that none of the Bell designs are capable of giving constant inlet conditions to the runner at part or overload, the article emphasises that the superiority of the Sfindex design lies in the new guide-apparatus patented by this firm. It is claimed that the improvements covered in the Sfindex patents give high efficiency at part load, thus solving for the first time a long-standing problem in the development of full-admission impulse turbines. (*Technische Rundschau*, Bern, Vol. 46, No. 28, 2.7.54, 4 cols., 9 ff.)

Induced Rainfall

In this paper, first submitted to the Société Hydrotechnique de France on March 18, 1953, the author analyses the conditions prerequisite to rainfall, either in settled or shower form, and then approaches the problem of induced rainfall. The various methods applied to rainmaking in Europe and U.S.A. are briefly reviewed. It is obvious that induced rainfall is not, as is sometimes believed, a miraculous panacea which can transform deserts into grazing grounds and the artificial production of the right sort of rain-precipitating clouds is still out of the realm of possibility, but we do now know that when the meteorological conditions necessary to natural rainfall are available, precipitation can be induced or hastened. It is even possible to induce a precipitation which would not have developed in the natural way such as, for example, in the instance of cloud limitations in altitude; on the other hand, it does not appear possible to control in any way precipitation from cloud systems producing naturally settled and abundant rain, while shower clouds are much more vulnerable to seeding. Even reduced to its true proportions, the problem of induced precipitation still offers a wide field of possibilities for researches which, up to the present, have probably not been carried out boldly enough. (J. Roulleau, *La Houille Blanche*, Vol. 9, No. 1, p. 38, 5 pp.)

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ENGINEERING DESIGNER, aged about 35 years, required with good mechanical and structural experience for work on tender designs for crane and sluice installations. House available Ipswich or Felixstowe. Apply stating age, technical training and experience to Secretary, Ransomes & Rapier Ltd., Ipswich.